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Full-Scale Accelerated Testing of Multi-axial Geogrid Stabilized Flexible Pavements

W. Jeremy Robinson, Jeb S. Tingle, and Gregory J. Norwood   June 2017

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Final report

Abstract

The U.S. Army Engineer Research and Development Center (ERDC) constructed a full-scale test section to evaluate the performance of geogrid-stabilized highway pavements. The test section consisted of two pavements composed of 3 in. of hot mix asphalt over a crushed limestone base course over a 6 CBR subgrade. One section was stabilized with Tensar's TX5 multi-axial geogrid, and the second section was stabilized with a developmental geogrid referred to as TX8. Each section was instrumented with moisture probes, pore-water pressure probes, temperature probes, earth pressure cells, and asphalt strain gauges. Simulated truck traffic was applied using a heavy vehicle simulator (HVS) to evaluate the rutting performance and collect instrumentation response data. This report summarizes the material characterization and construction techniques, instrumentation response, and performance response of each section under simulated truck traffic. Comparisons are drawn to previous sections constructed and trafficked at ERDC without geogrids utilizing similar construction techniques and material properties.

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Preface

This study was conducted for Tensar International Corporation located at Alpharetta, Georgia. Dr. Mark H. Wayne, Tensar, provided technical guidance and review during the project. Mr. Jeb S. Tingle was the ERDC program manager.

The work described in this report was performed by the Airfields and Pavements Branch (GMA) of the Engineering Systems and Materials Division (GM), U.S. Army Engineer Research and Development Center, Geotechnical and Structures Laboratory (ERDC-GSL). At the time of publication, Dr. Timothy W. Rushing was Chief, CEERD-GMA; Dr. Gordon W. McMahon was Chief, CEERD-GM; and Ms. Pamela G. Kinnebrew, CEERD-GZT, was Technical Director for Military Engineering. The Deputy Director of ERDC-GSL was Dr. William P. Grogan, and the Director was Mr. Bartley P. Durst.

COL Bryan S. Green was the Commander of ERDC, and Dr. David W. Pittman was the Director.

Unit Conversion Factors

Multiply	By	To Obtain
cubic feet	0.02831685	cubic meters
cubic inches	1.6387064 E-05	cubic meters
cubic yards	0.7645549	cubic meters
degrees Fahrenheit	(F-32)/1.8	degrees Celsius
feet	0.3048	meters
gallons (U.S. liquid)	3.785412 E-03	cubic meters
inches	0.0254	meters
miles per hour	0.44704	meters per second
mils	0.0254	millimeters
pounds (force)	4.448222	newtons
pounds (force) per foot	14.59390	newtons per meter
pounds (force) per inch	175.1268	newtons per meter
pounds (force) per square foot	47.88026	pascals
pounds (force) per square inch	6.894757	kilopascals
pounds (mass)	0.45359237	kilograms
pounds (mass) per cubic foot	16.01846	kilograms per cubic meter
pounds (mass) per cubic inch	2.757990 E+04	kilograms per cubic meter
pounds (mass) per square foot	4.882428	kilograms per square meter
pounds (mass) per square yard	0.542492	kilograms per square meter
square feet	0.09290304	square meters
square inches	6.4516 E-04	square meters
tons (force)	8,896.443	newtons
tons (force) per square foot	95.76052	kilopascals
yards	0.9144	meters

1   Introduction

1.1   Background

With increased material costs and reduced budgets, transportation officials are often tasked with applying innovative solutions to pavement design and construction projects. The inclusion of geosynthetics in pavement structures has been utilized as a means of reducing costs and/or extending pavement service life. These products have been shown to be effective in overcoming challenging subsurface conditions, in reducing overall pavement thickness, and in reducing labor and equipment requirements. As new geosynthetic products are developed, full-scale performance data are needed to provide quantifiable benefits to pavement designers.

1.2   Objective

The objective of this effort was to construct and traffic full-scale flexible pavement sections to provide performance data for comparing the effectiveness of Tensar's multi-axial geogrids to an unstabilized flexible pavement section. The test items constructed were similar to previously tested unstabilized and stabilized flexible pavement sections allowing for performance comparisons. A secondary objective was to record pavement response data for the verification of mechanistic performance models for Tensar's geogrid stabilization products currently on the market and for the validation of the SpectraPave4Pro™ software used to design the test items.

1.3   Scope

The scope of this project included the construction and trafficking of a full-scale test section comprised of a geogrid-stabilized base course. The U.S. Army Engineer Research and Development Center (ERDC) constructed the full-scale test section as designed by Tensar under shelter in its Hangar 2 Pavement Test Facility. During construction, quality control data were collected to verify layer properties and to ensure that the project objectives were accomplished. The test items were trafficked with ERDC's Heavy Vehicle Simulator (HVS). Pavement performance data were collected at selected intervals during trafficking for use in comparing the performance of the test items with the geogrid-stabilized base course to the historical test item with an unstabilized base course. In addition, instrumentation response data were collected and reported to support the verification of mechanistic pavement performance models.

2   Test Plan and Layout

The structural design of the test section was developed by Tensar Corporation. The test items were constructed utilizing similar materials and construction techniques as previous unstabilized and stabilized sections (Norwood and Tingle 2014) to extend the usefulness of performance comparisons. A profile view of the stabilized and unstabilized test section (Norwood and Tingle 2014) is shown in Figure 1. Each test item consisted of a 10-ft-wide by 50-ft-long testing area. For this study, two test items were constructed with each item stabilized with a different multi-axial geogrid. Item 1 was stabilized with TX5 multi-axial geogrid, and Item 2 was stabilized with a developmental grid identified as TX8. The TX5 multi-axial geogrid was previously tested but due to a temperature control malfunction in the HVS, temperatures rose outside the established control range and the section was reconstructed and retested. The subgrade for the test items was constructed with nominal 36 in. of high-plasticity clay (CH) placed to achieve the target California Bearing Ratio (CBR) value of 6%. A CBR of 6% was selected to provide a test section with moderate subgrade strength. The 6-in.-thick aggregate base course consisted of a crushed limestone, stabilized by placing a multi-axial geogrid at the base-subgrade interface. The test-section surface consisted of a 3-in.-thick hot mix asphalt (HMA) layer. All construction and traffic testing occurred under ERDC's Hangar 2 pavement testing facility, minimizing the potential for moisture variations due to environmental factors.

Figure 1. Test section profile view.
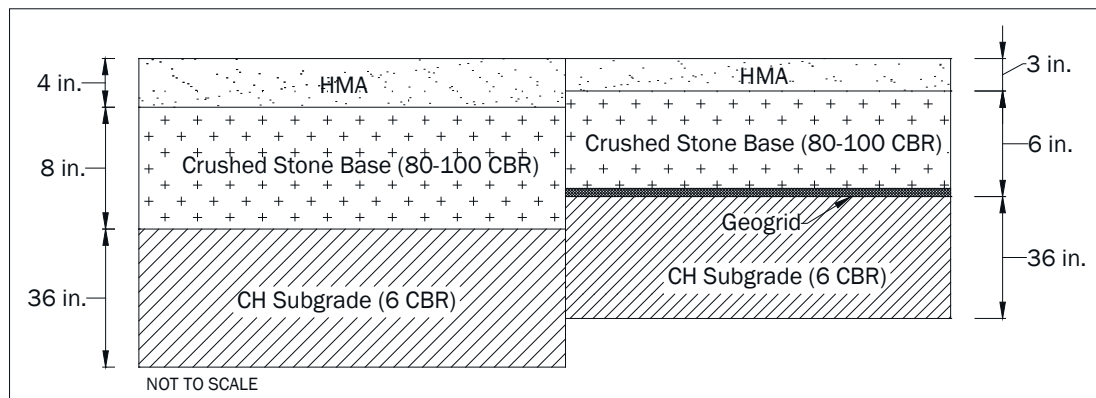
The structural design of the test section was developed by Tensar Corporation following the guidelines set forth in the American Association of State Highway and Transportation Officials (AASHTO) 1993 design guide (AASHTO 1993) and their SpectraPave4Pro™ software. The geogrid-stabilized items were assigned a structural number of 2.92 with an estimated service life of 580,000 equivalent single axle loads (ESALs).

The test items contained a suite of instrumentation consisting of asphalt strain gauges, earth pressure cells, single-depth deflectometers, moisture probes, pore-water pressure transducers, and temperature probes.

The test item was trafficked with ERDC's Heavy Vehicle Simulator (HVS) simulating normal highway loadings. The configuration used for testing consisted of a tandem-axle dual wheel gear loaded to a nominal load of 20,000 lb. Tire pressures were maintained at 120 psi throughout testing. The equivalent axle load factor for this configuration is 2.08. Therefore, one pass of the HVS was equal to 2.08 equivalent standard axle loads (ESALs).

The traffic loading was applied at a target test chamber temperature of 77° Fahrenheit, with an allowable temperature range of 72-82° Fahrenheit. The HVS is equipped with automated software that halts traffic and alerts the operator if the temperature falls outside the prescribed range.

3 Materials

Laboratory tests were performed to characterize each component layer material. The material characterization results are presented in the following paragraphs.

3.1 Subgrade

A locally sourced high-plasticity clay was used to construct the nominal 36-in.-thick test-section subgrade. A particle size analysis, shown in Figure 2, indicated the material consisted of 96.8% fines passing the No. 200 sieve. The soil had a liquid limit (LL) of 65%, a plastic limit (PL) of 27%, and a plasticity index (PI) of 37%, as determined by ASTM D4318. According to the Unified Soil Classification System (USCS), the soil was classified as a high-plasticity clay (CH) and an A-7-6 according to the American Association of State and Highway Transportation Officials (AASHTO) classification system.

Figure 2. Subgrade particle-size analysis.
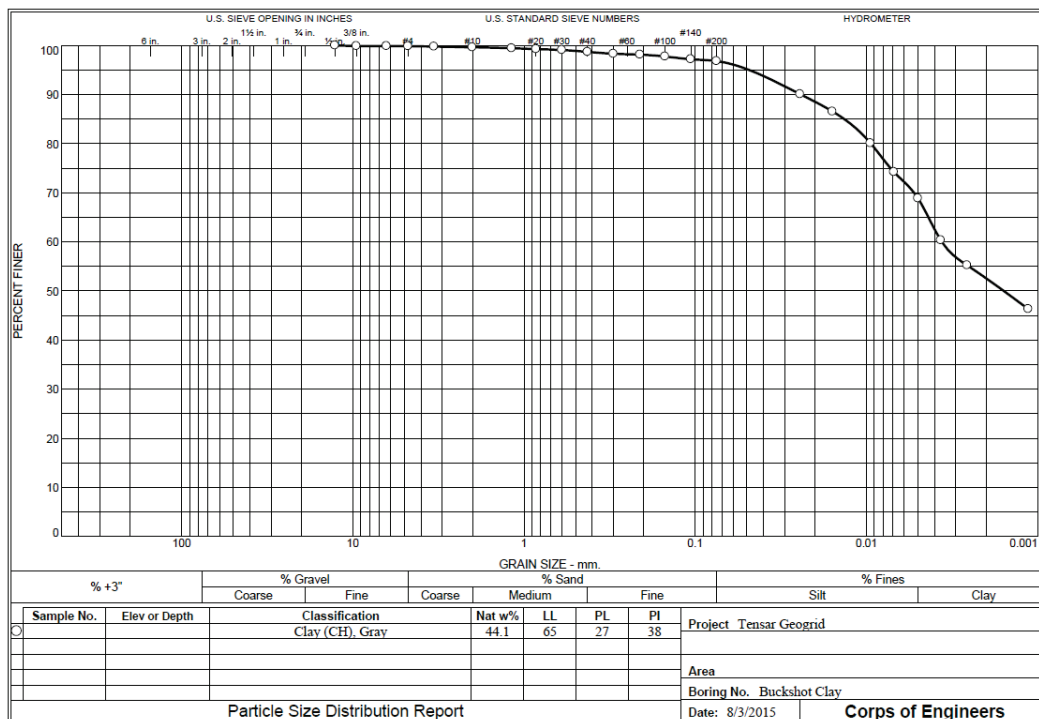
Modified proctor compaction tests (ASTM D1557) were performed to determine the relationship between moisture content and dry density. Maximum dry density was found to be 101.8 pcf at an optimum moisture content of 18.6%. Graphical results of the moisture-density relationship test are shown in Figure 3.

Figure 3. Clay subgrade moisture-density relationship.
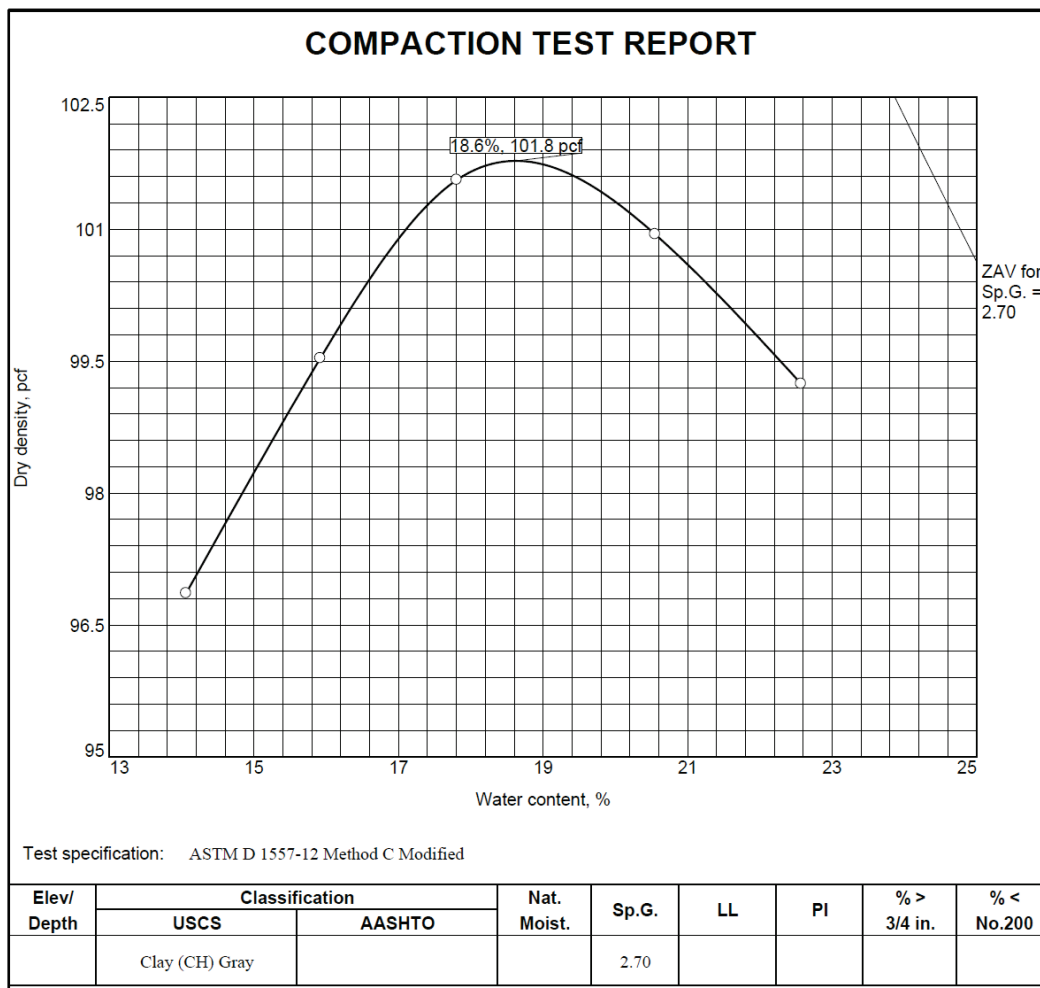
To determine an in-place moisture content at the targeted 6 CBR, a suite of laboratory CBR tests (ASTM D1883) were performed. These tests were conducted at moisture contents ranging from approximately 17% to approximately 40%. A target in-place moisture content of 33% was selected based upon the relationship between moisture content and CBR. The relationship between CBR and moisture content is presented graphically in Figure 4.

Figure 4. Clay subgrade CBR/moisture content relationship.
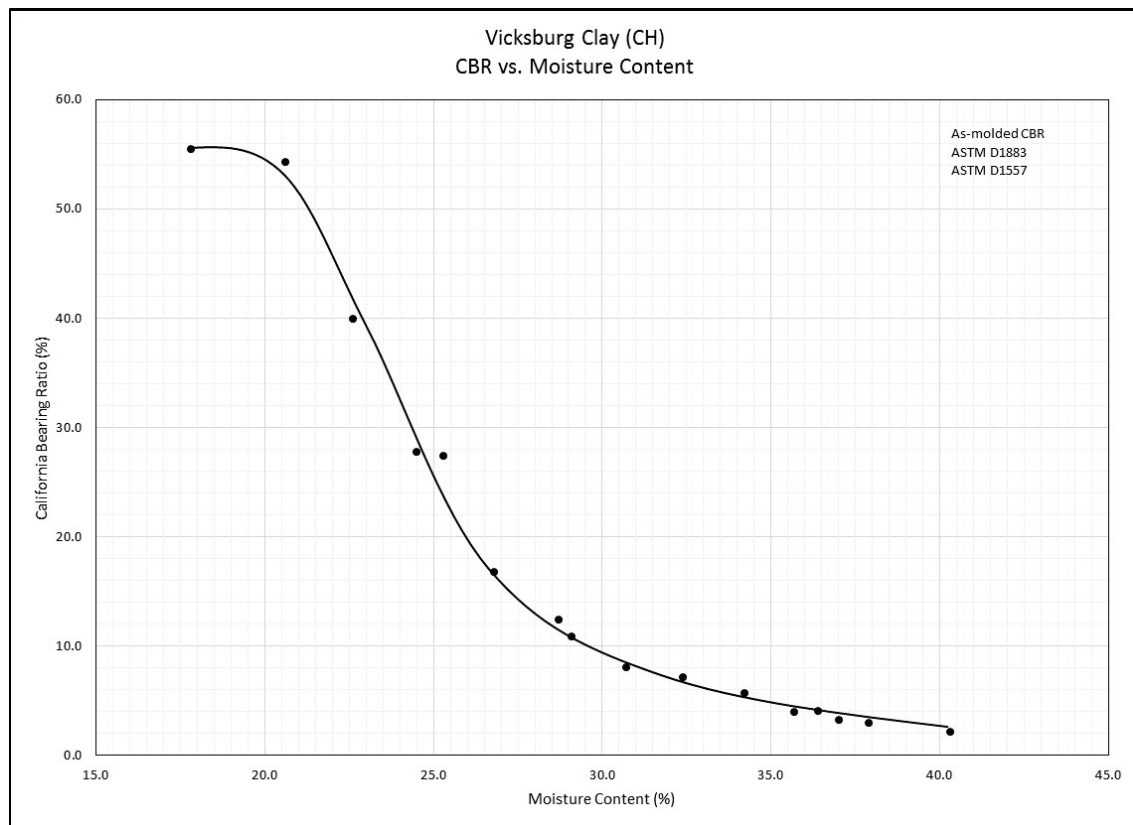
3.2   Base course

Locally available crushed limestone was used to construct the flexible aggregate base course. The gradation for the crushed limestone is shown in Figure 5. ASTM procedure D2487 was used to determine that the base course was comprised of 71.5% gravel, 23.3% sand, and 5.2% non-plastic fines passing the No. 200 sieve. The coefficient of curvature (Cc) was calculated as 2.71, and the coefficient of uniformity (Cu) was 21.16. The crushed limestone aggregate base was classified as a well-graded gravel with silt and sand (GW-GM) according to the USCS and an A-1-a according to the AASHTO procedure. Modified proctor compaction tests (Figure 6) were performed in accordance with ASTM D1557 Method C Modified. The maximum dry density was 146.7 pcf at an optimum moisture content of 5.9%.

Figure 5. Crushed limestone base particle-size analysis.
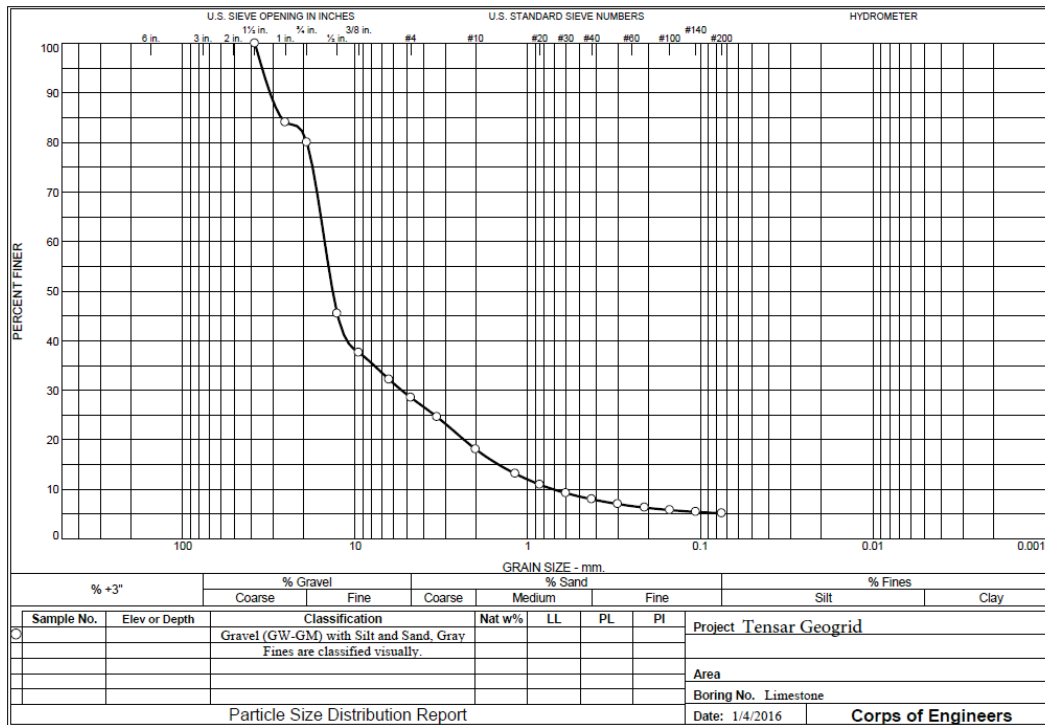
Figure 6. Limestone base moisture/density relationship.
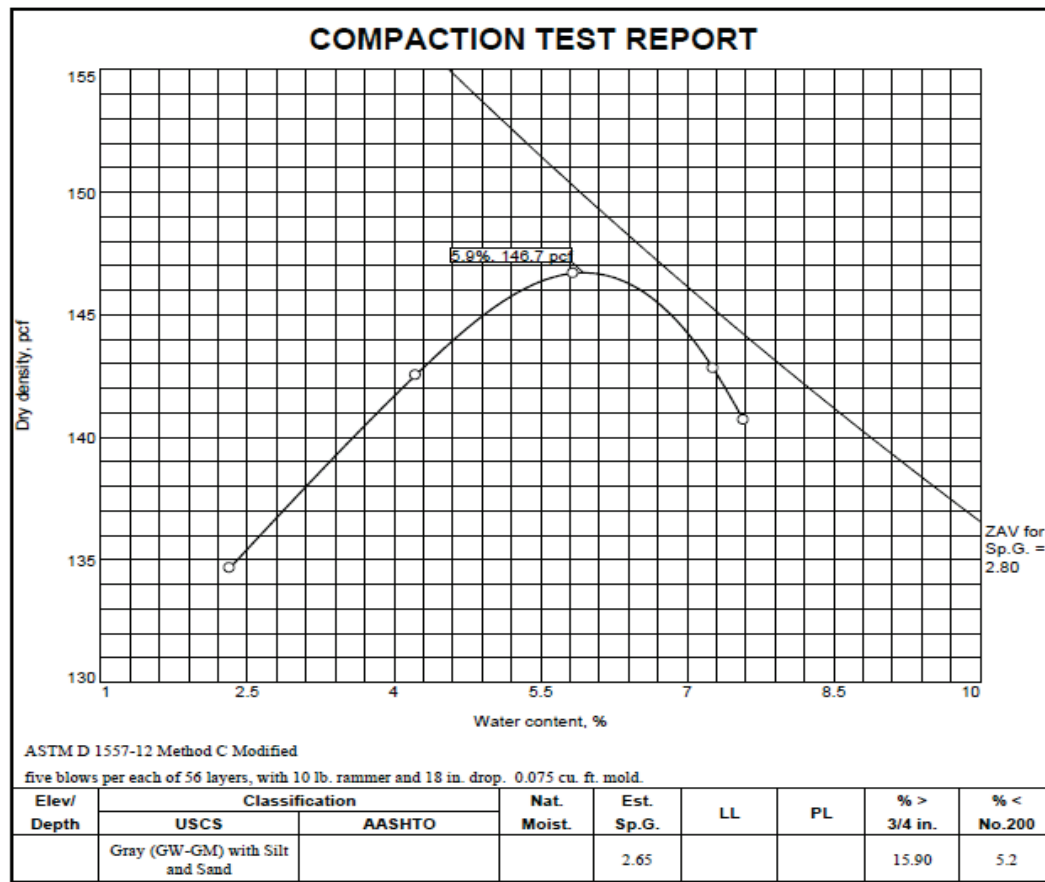
3.3 Asphalt

A 9.5-mm nominal maximum aggregate size (NMAS) HMA surface mixture was selected for placement of the final wearing surface on both test items. The asphalt mixture was one that is representative of the local highway mix used on medium-traffic-level roadways in Mississippi. Mixture properties were adjusted to closely replicate those used in previous geogrid evaluation studies. Pertinent mixture properties, including control values (Norwood and Tingle 2014) are presented in Table 1.

Table 1. Hot-mix asphalt properties.

Test Property		Item 1 & Item 2	Control 4-in. Nominal AC Unstabilized Base (Norwood and Tingle 2014)
N_{design}		75	75
Binder Grade		PG 67-22	PG 67-22
Mixing Temp (°F)		310	310
Compaction Temp (°F)		300	292
Percent Passing (%)	1.0 in. (25.0 mm)	100	100
	3/4 in. (19.0 mm)	100	100
	1/2 in. (12.5 mm)	100	100
	3/8 in. (9.5 mm)	93	95
	#4 (4.75 mm)	57	54
	#8 (2.36 mm)	38	34
	#16 (1.18 mm)	27	27
	#30 (0.60 mm)	22	20
	#50 (0.30 mm)	12	7
	#100 (0.15 mm)	7	5
	#200 (0.075 mm)	5.1	3.8
RAP (%)		20	15
RAP AC (%)		5.0	5.5
G_{sb}		2.510	2.6
P_b (%)		5.7	5.7
G_{mm}		2.361	2.429
VMA (%)		15.1	15.4
VFA (%)		73.5	74.0
P_{200}/P_{be}		1.02	0.76

3.4 Geogrids

The geogrids evaluated in this study were both multi-axial geogrid products manufactured from a punched and drawn polypropylene sheet. The geogrids have a hexagonal structure forming a series of triangular apertures. Nominal index properties provided for each geogrid are presented in Table 2.

Table 2. Geogrid physical properties.

Index Property	TX5 (Item 1)			TX8 (Item 2)		
	Longitudinal	Diagonal	Transverse	Longitudinal	Diagonal	Transverse
Rib pitch, mm (in.)	40 (1.60)	40 (1.60)	--	33 (1.30)	33 (1.30)	--
Mid-rib depth, mm (in.)	--	1.3 (0.05)	1.2 (0.05)	--	1.6 (0.06)	1.2 (0.05)
Mid-rib width, mm (in.)	--	0.9 (0.04)	1.2 (0.05)	--	0.4 (0.02)	0.7 (0.03)

4 Instrumentation

Instrumentation was installed in the subgrade, base course, and HMA surface course to monitor pavement response during test-section trafficking. Sensors used to capture the pavement response included earth pressure cells (EPCs), single-depth deflectometers (SDDs), and asphalt strain gauges (ASGs). Pore-water pressure, temperature sensors, and moisture sensors were installed to monitor environmental parameters. Figure 7 and Figure 8 show the plan and profile view of the typical instrumentation layout for each traffic lane, respectively.

4.1 Earth pressure cells

Vertical stresses in the base course and subgrade were measured using 9-in.-diam Geokon EPCs. EPCs provide a quantitative measurement of the vertical distribution of the stresses within each traffic lane during testing. Cells with a maximum pressure range of 100 psi were installed in the subgrade, and 200-psi EPCs were installed in the base course. The EPCs in the base course were located 2 in. below the top of the base, while the EPCs placed in the subgrade were located 2 in. below the top of the subgrade. Figure 9 shows an EPC being installed 2 in. below the surface of the subgrade at the interface with the base course.
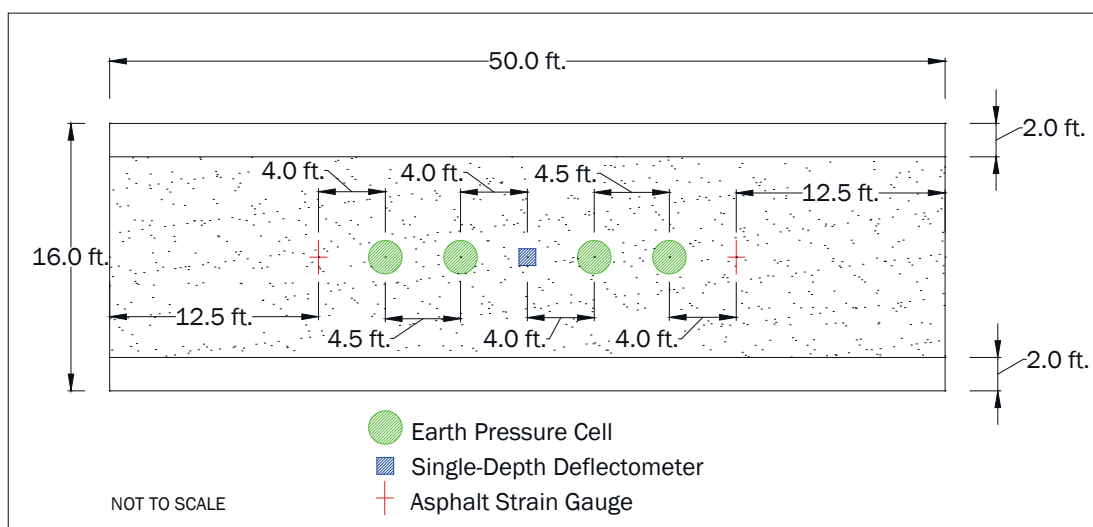
Figure 7. Plan view of instrumentation layout.

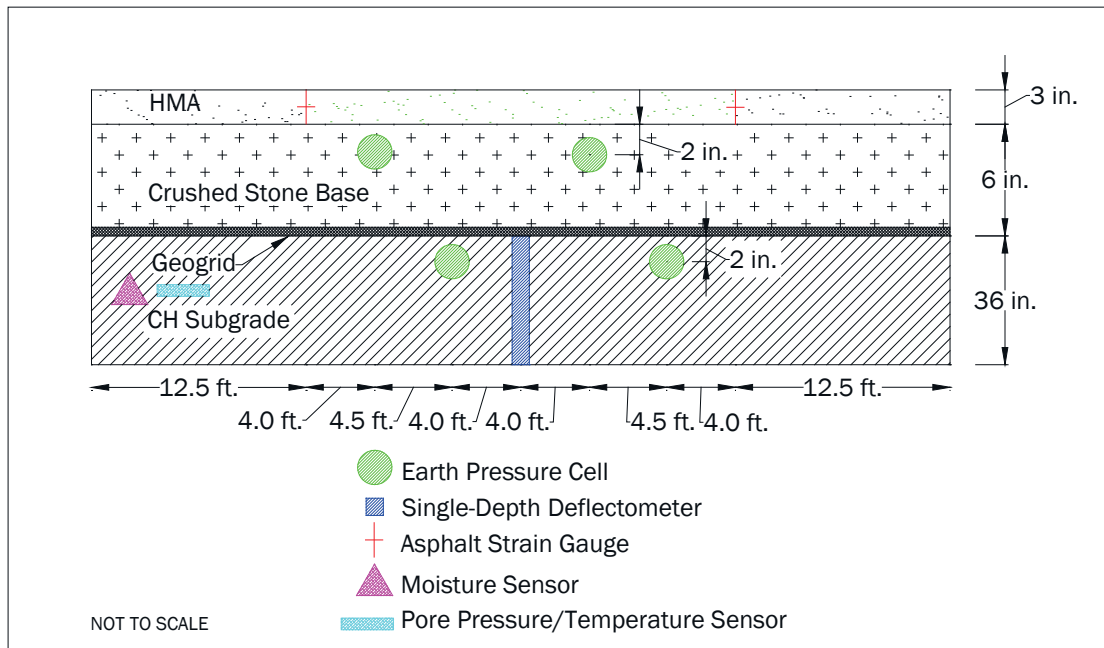Figure 8. Profile view of instrumentation layout.

Figure 9. Earth pressure cell installation in subgrade.

4.2   Asphalt strain gauges

Tensile strain at the bottom of an HMA layer provides a quantitative measure of the pavement response during trafficking. The tensile strain at the bottom of the asphalt is a key response parameter linked to fatigue damage in the HMA layer. For this study, strain at the bottom of the HMA surface was measured using dynamic asphalt strain gauges (ASGs) in both the transverse and longitudinal directions. The ASGs were manufactured by Tokyo Sokki and are capable of measuring a range of ±5,000 microstrain. The gauges were adhered to the surface of the base course with a heated asphalt binder, and HMA from the asphalt paver was placed as cover over each of the gauges immediately prior to paving of the entire test section. This process is shown in Figure 10 and Figure 11, respectively.

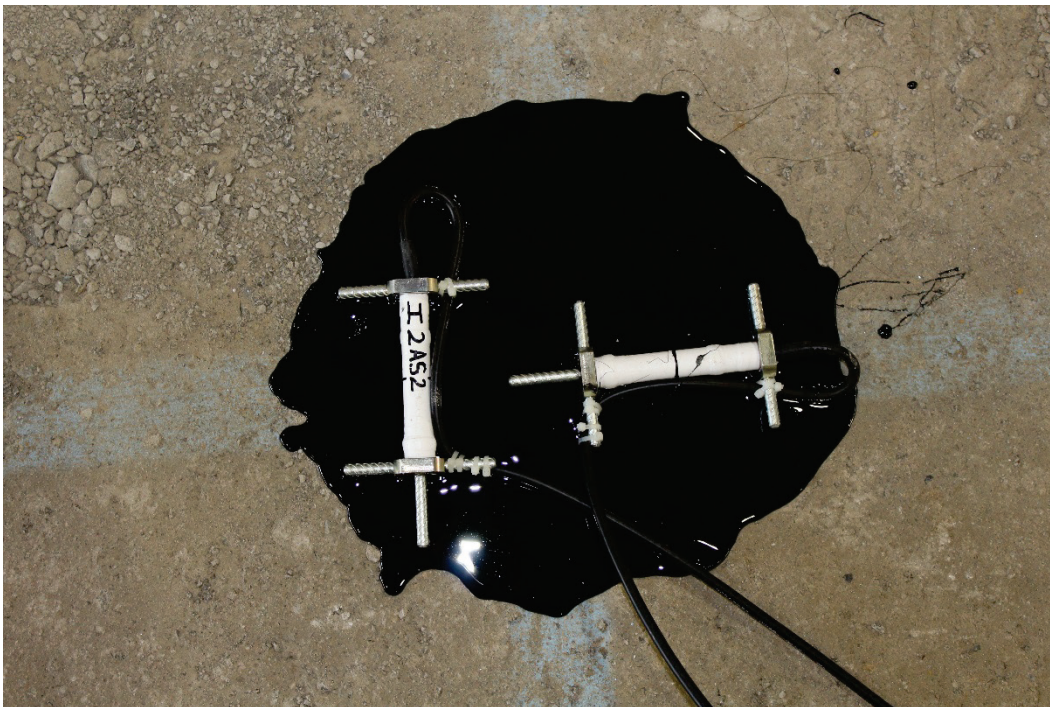Figure 10. Asphalt strain gauge adhered to crushed stone base.

Figure 11. Asphalt strain gauge covered with HMA prior to paving.

4.3   Single-depth deflectometer

Measurements of deformations in the subgrade are used to quantify the
stabilization benefits of a geogrid as well as assisting with the validation of
the failure mechanism within the pavement structure. Potential reductions
in vertical stresses at the subgrade should also be reflected as reductions in
the deformation. In theory for pavements with similar structural com-
ponents, the geogrid-stabilized pavement should show lower deflections
than the unstabilized pavement at a given traffic level.

Vertical deflections in the subgrade were measured using single-depth
deflectometers (SDDs). One SDD was placed in the middle of each test
item along the centerline of traffic. The SDD was placed such that the shaft
was anchored at a depth of 8 ft from the top of the subgrade. A linear
velocity displacement transducer (LVDT) with a range of ±2 in. was placed
in the housing such that it was in contact with both the anchor rod and the
surface plate as shown in Figure 12. Thus, the LVDT measured movement
of the plate 2 in. below the base-subgrade interface relative to the control
point located at a depth of 8 ft.

Figure 12. Single-depth deflectometer installation.

5 Pavement Characterization

5.1 As-built properties

Quality control tests were performed during construction of each material lift to ensure target values were achieved and to monitor material consistency. Dry density and moisture content were measured using a nuclear moisture density device in accordance with ASTM D6938-15 to verify the uniformity of each material lift. Field in-place CBR tests were performed in general accordance with ASTM D4429-09a on each compacted lift to ensure target values were achieved. To further characterize the strength of the completed base and subgrade layers, Dynamic Cone Penetrometer (DCP) tests were performed in accordance with ASTM D6951-09. Asphalt cores were obtained from each test item, and core densities were determined in accordance with AASHTO T166.

As-built material properties of the base and subgrade are summarized in Table 3. Also included are the as-built control section properties from a previous test-section construction that included a comparable unstabilized control section (Norwood and Tingle 2014).

Table 3. As-built section properties.

Property	Item 1 TX5 Stabilized Base	Item 2 TX8 Stabilized Base	Control 4-in. Nominal AC Unstabilized Base (Norwood and Tingle 2014)
CH Subgrade Properties			
Wet Density (pcf)	116.5 ± 1.0	116.6 ± 1.0	116.9
Dry Density (pcf)	88.5 ± 1.1	88.7 ± 1.2	88.9
Nuclear Moisture Content (%)	31.6 ± 0.8	31.6 ± 1.1	30.0
Oven-Dried Moisture (%)	32.7 ± 1.3	32.6 ± 1.2	34.1
In-Place CBR (%)	5.9 ± 0.2	5.9 ± 0.2	5.9
Crushed Limestone Properties			
Wet Density (pcf)	138.8 ± 1.6	141.5 ± 1.8	143.0
Dry Density (pcf)	134.5 ± 1.5	137.2 ± 1.7	138.1
Nuclear Moisture Content (%)	3.3 ± 0.1	3.3 ± 0.2	3.6
Oven-Dried Moisture (%)	3.0 ± 0.3	3.2 ± 0.2	2.4
In-Place CBR (%)	98.6 ± 9.1	100(+) ± 4.9	95.3
Thickness (in.)	5.82 ± 0.36	5.90 ± 0.33	7.7

Property	Item 1 TX5 Stabilized Base	Item 2 TX8 Stabilized Base	Control 4-in. Nominal AC Unstabilized Base (Norwood and Tingle 2014)
Hot-Mix Asphalt Properties			
Density (% of G_{mm})	93.4 ± 0.5	93.2 ± 1.0	--
Thickness (in.)	3.20 ± 0.21	3.22 ± 0.11	4.0*

*Nominal thickness reported

5.2   Dynamic cone penetrometer

A series of dynamic cone penetrometer (DCP) tests were performed to further characterize the strength of the unbound pavement layers. DCP tests were performed after construction of the base and subgrade layers on both sides of the traffic lane, following the procedures described by ASTM D 6951-09. Measured values of the DCP index (millimeters of penetration per hammer blow) were converted to CBR strength using the relationship developed by Webster et al. (1992, 1994). As-built DCP results are shown for Item 1 Station 25 West in Figure 13 and Item 2 Station 25 East in Figure 14.

5.3   Falling weight deflectometer

Falling weight deflectometer tests were performed on the surface of both test items after construction and prior to trafficking. The measured impulse stiffness modulus (ISM) was used to evaluate the stiffness of the constructed pavement section and provide a baseline for subsequent comparison under traffic. The ISM is the ratio of the applied load to the measured plate deflection with higher values representing a stiffer pavement structure. Additionally, the ISM values were compared to values obtained from previous test sections to verify the validity of performance comparisons. As-built ISM values for Item 1, Item 2, and the unstabilized control (Norwood and Tingle 2014) are presented in Figure 15. As-built ISM values for each test item were found to be relatively consistent along the length of each test item. Minor differences were observed in the test items with Item 2 having a slightly higher ISM value than Item 1. Both stabilized items were found to have lower ISM values than the unstabilized item. This was anticipated as the unstabilized item had an additional 2 in. of base course and 1 in. of HMA.

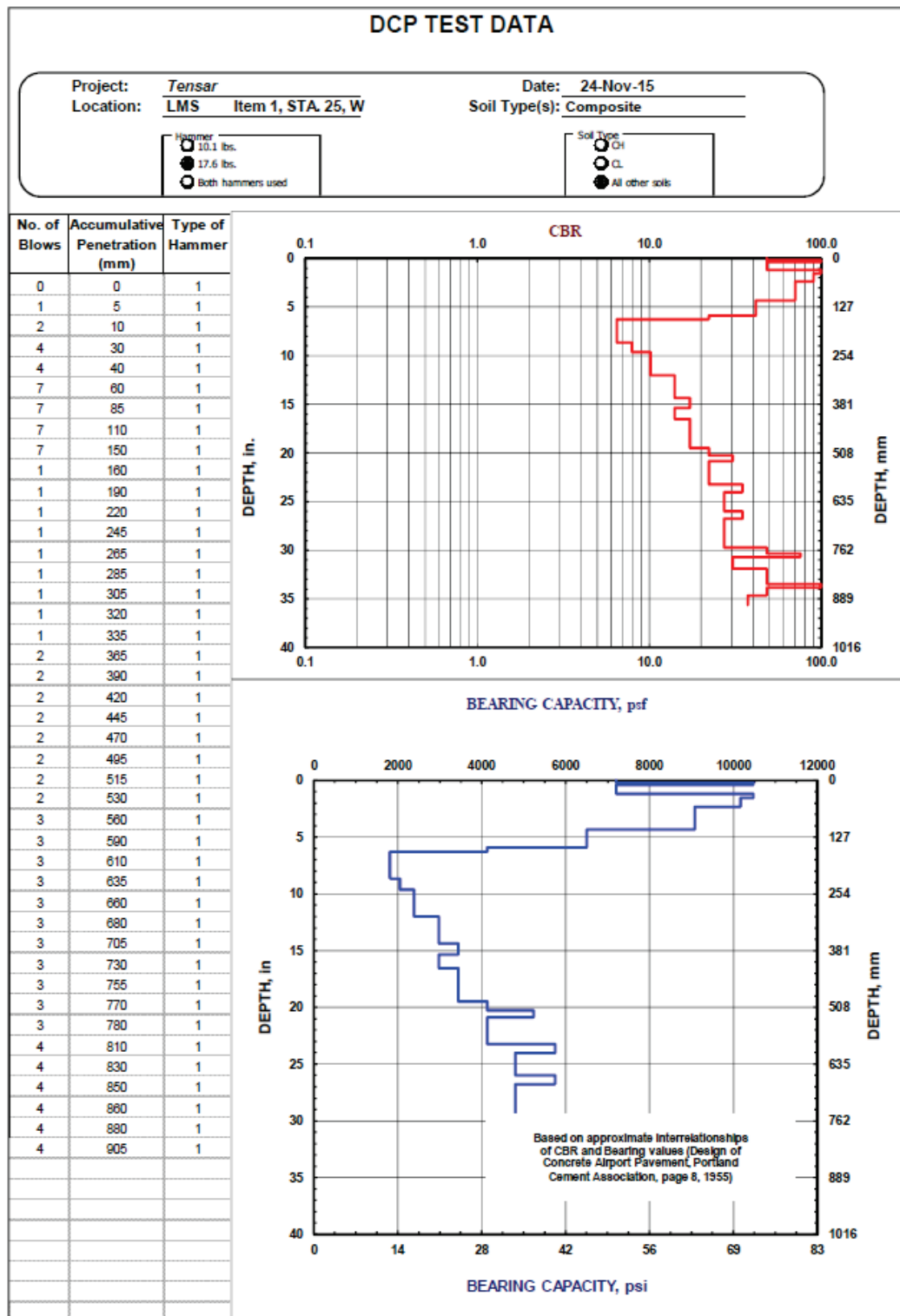Figure 13. As-built DCP results for Item 1 Station 25, West.

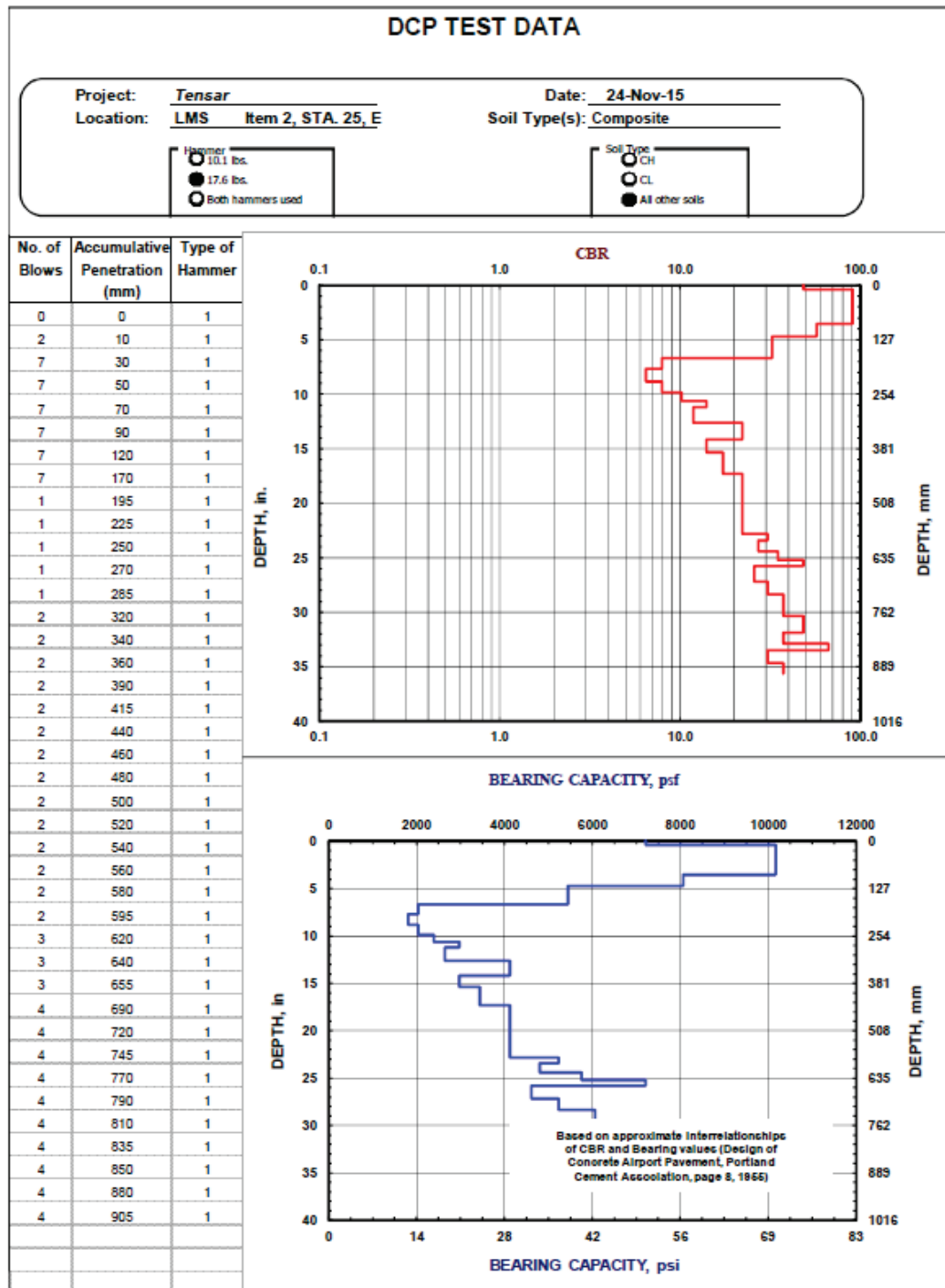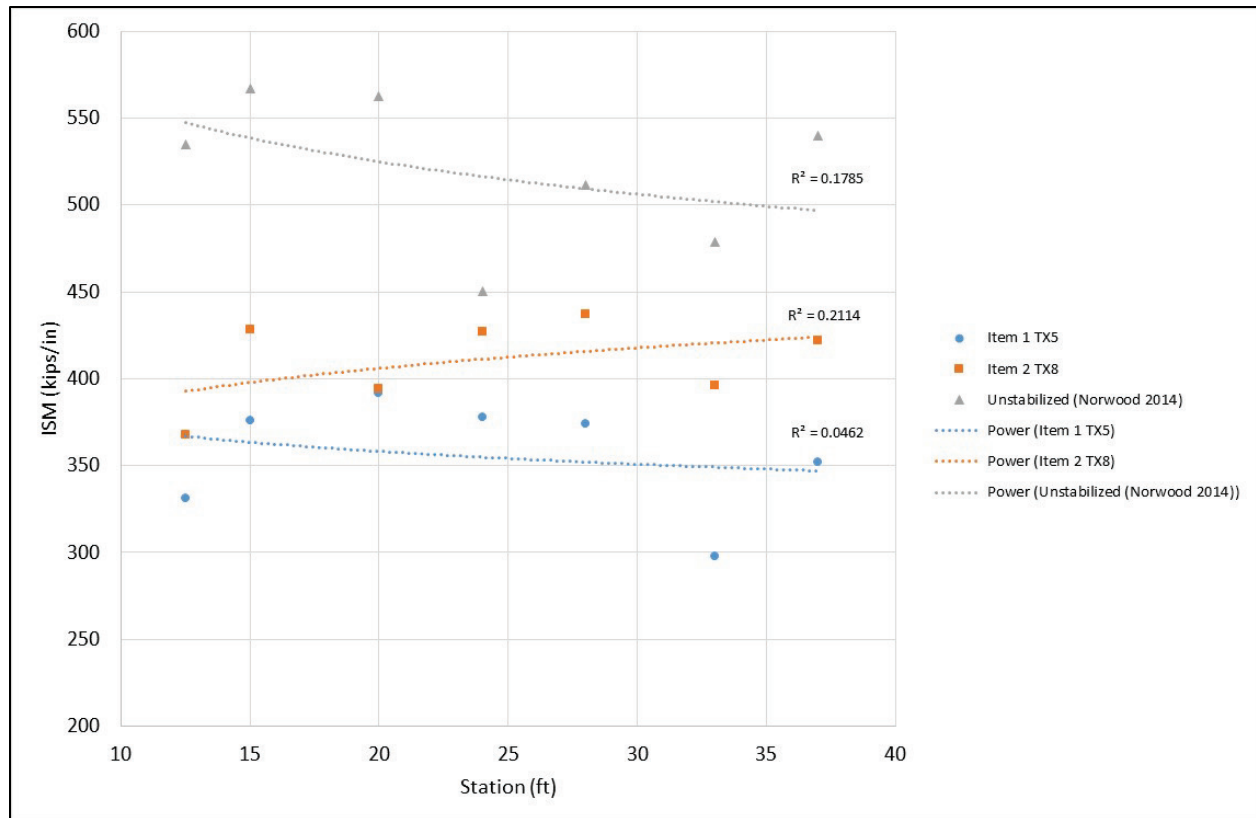Figure 14. As-built DCP results for Item 2 Station 25, East.

No. of Blows	Accumulative Penetration (mm)	Type of Hammer
0	0	1
2	10	1
7	30	1
7	50	1
7	70	1
7	90	1
7	120	1
7	170	1
1	195	1
1	225	1
1	250	1
1	270	1
1	285	1
2	320	1
2	340	1
2	360	1
2	390	1
2	415	1
2	440	1
2	460	1
2	480	1
2	500	1
2	520	1
2	540	1
2	560	1
2	580	1
2	595	1
3	620	1
3	640	1
3	655	1
4	690	1
4	720	1
4	745	1
4	770	1
4	790	1
4	810	1
4	835	1
4	850	1
4	880	1
4	905	1

Figure 15. As-built ISM values.

6   Traffic Testing

Traffic testing on both test items was performed utilizing a dual-wheel tandem axle gear (Figure 16) on the ERDC HVS. The gear configuration was loaded to a 20,000-lb nominal load and was verified prior to testing by weighing each axle with portable aircraft wheel scales. Tire pressures were monitored and maintained at 120 psi throughout traffic testing. The equivalent single axle load factor for this configuration was 2.08; therefore, each pass of the HVS was equal to 2.08 equivalent single axle loads (ESALS). The environmental chamber was maintained at a target pavement temperature of 77°F with an allowable range of ±5°F during trafficking to minimize the effect of temperature variation on rutting performance.

Each test item was trafficked utilizing a normally distributed bi-directional traffic pattern shown in Figure 17. The lateral offset indices refer to 1-in. increments along which the axle shifts laterally, resulting in a total lateral wander of approximately 3 feet. A 3-ft-wide wander pattern was utilized to mimic previous studies and to duplicate observations in other test sections.

Figure 16. Dual-wheel tandem axle configuration.

Figure 17. Normally distributed traffic pattern.
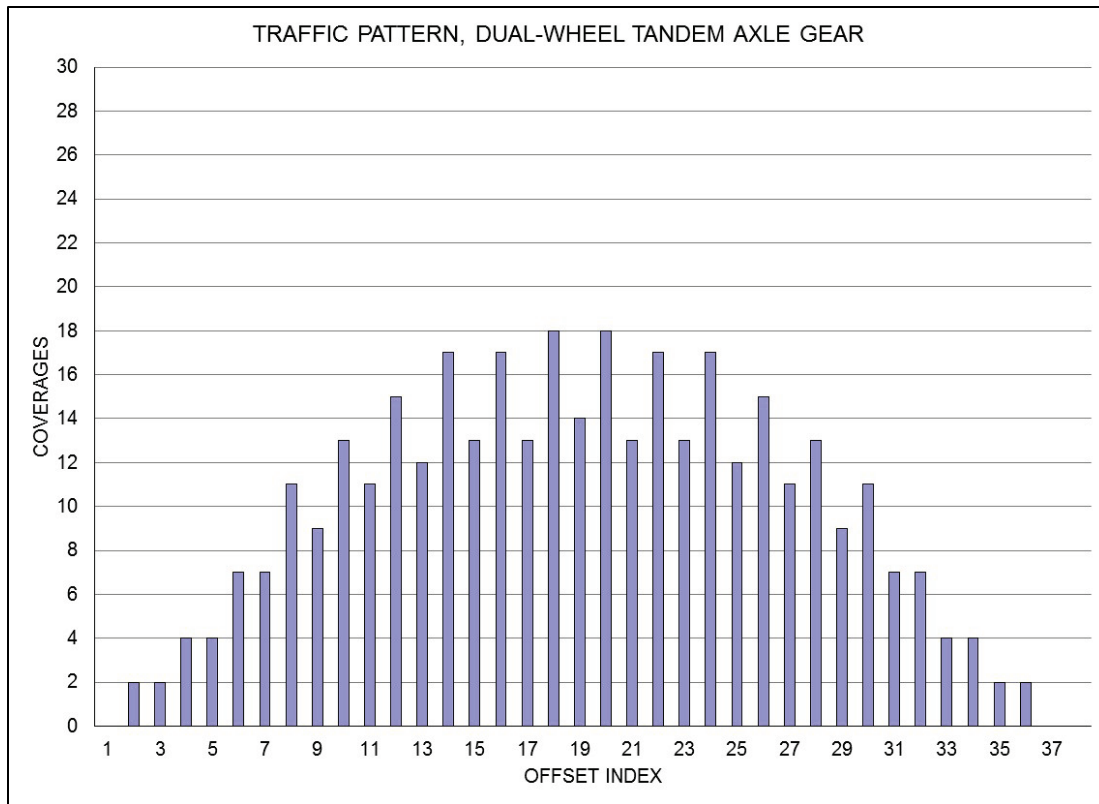
7 Results

Failure of the flexible pavement test items was defined as 1 in. of rutting. This failure definition was based on the concept of pavement serviceability and the fact that pavement serviceability begins to decrease exponentially in flexible pavements after 1 in. of rutting. Results are presented to document the changes in vertical alignment and instrumentation response at each traffic interval. Included within each results section are the responses documented from the unstabilized control section presented by Norwood and Tingle (2014).

7.1 Surface deformations

Surface deformation and rut depth measurements were made at predetermined traffic levels throughout the testing period. Surface deformation measurements represent changes in elevation and were made along the centerline and designated cross-sections of each test item. Surface deformation measurements were made utilizing robotic total station equipment to an accuracy of 0.08 in. Rut depth measurements were made by placing a metal straight edge across the traffic lane at designated cross sections and measuring the maximum rut depth using a machined depth wedge to an accuracy of 1/16 in. The maximum rut depth includes permanent surface deformation and upheaval along the edge of the traffic pattern. A schematic of the maximum rut depth measurement procedure is shown in Figure 18.

Figure 18. Measurement of maximum rut depth.
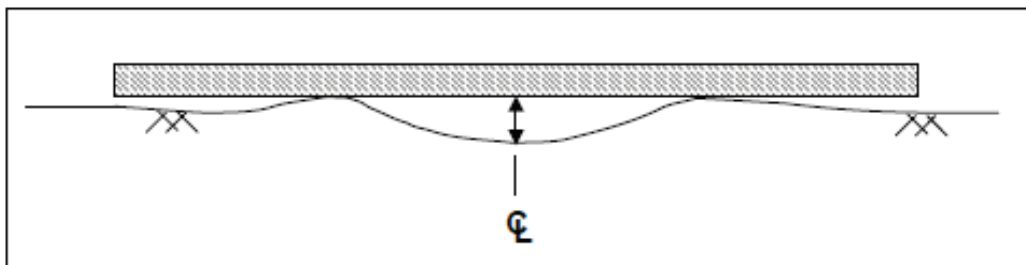
Maximum rut depths are presented in Figure 19 and Table 4. These values represent the maximum rut depth of 50% of the traffic lane. Maximum rut depth measurements were taken at stations 12.5, 18.75, 25.0, 31.25, and 37.5 of each test item, and the average of the maximum rut depth

measurements was reported. The test results indicate the stabilized test items displayed approximately 50-60% less rutting than the unstabilized control item despite the reduced base and surface layer thicknesses of the stabilized sections compared to the control section. Minimal differences in rutting were observed in Item 1 and Item 2, with Item 2 outperforming Item 1 at most traffic levels.

Surface deformation measurements presented in Figure 20 and Table 5 were obtained by averaging elevation measurements taken at 1-ft increments along the centerline of the trafficked area. Measurements made at the outer 5 feet of the traffic centerline were excluded from the averaging calculations due to the potential for non-representative deformations caused by the change of direction of the HVS carriage. The test results indicate that the stabilized test items displayed approximatley 30-35% less surface deformation than the unstabilized control item. Minimal differences in surface deformation were observed between Item 1 and Item 2, with Item 2 outperforming Item 1 at most traffic levels.

It should be noted that while a reduction in rutting and permanent deformation were observed in the stabilized items when compared to the unstabilized item, the overall magnitude of differences in the distress would be considered relatively small in practice.

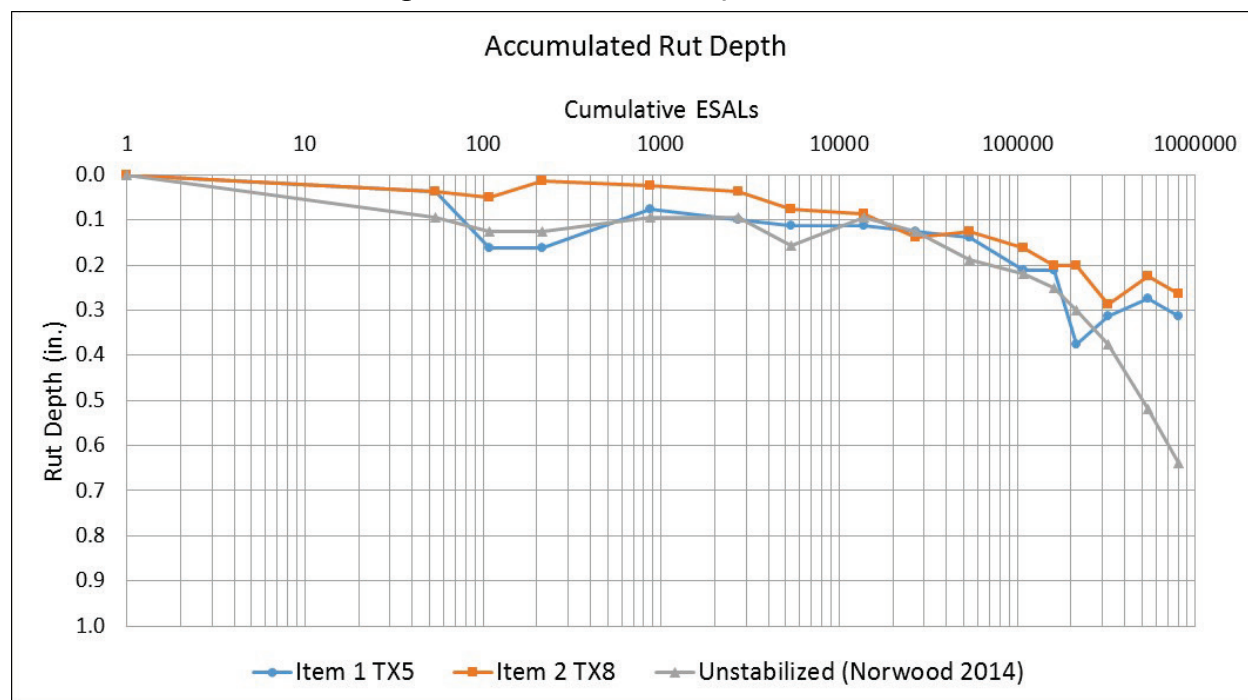Figure 19. Accumulated rut depth with traffic.

Table 4. Maximum rut depth measurements.

Test Item	Pavement Structure	ESALS								
		54	108	216	865	27,040	54,080	162,240	324,480	811,200
Item 1	3-in. HMA 6-in. base stabilized, TX5	0.038	0.163	0.163	0.075	0.125	0.138	0.213	0.313	0.313
Item 2	3-in. HMA 6-in. base stabilized, TX8	0.038	0.050	0.013	0.025	0.138	0.125	0.200	0.288	0.263
Control	4-in. HMA 8-in. base unstabilized	0.094	0.125	0.125	0.094	0.125	0.188	0.250	0.375	0.640

Figure 20. Permanent surface deformation with traffic.
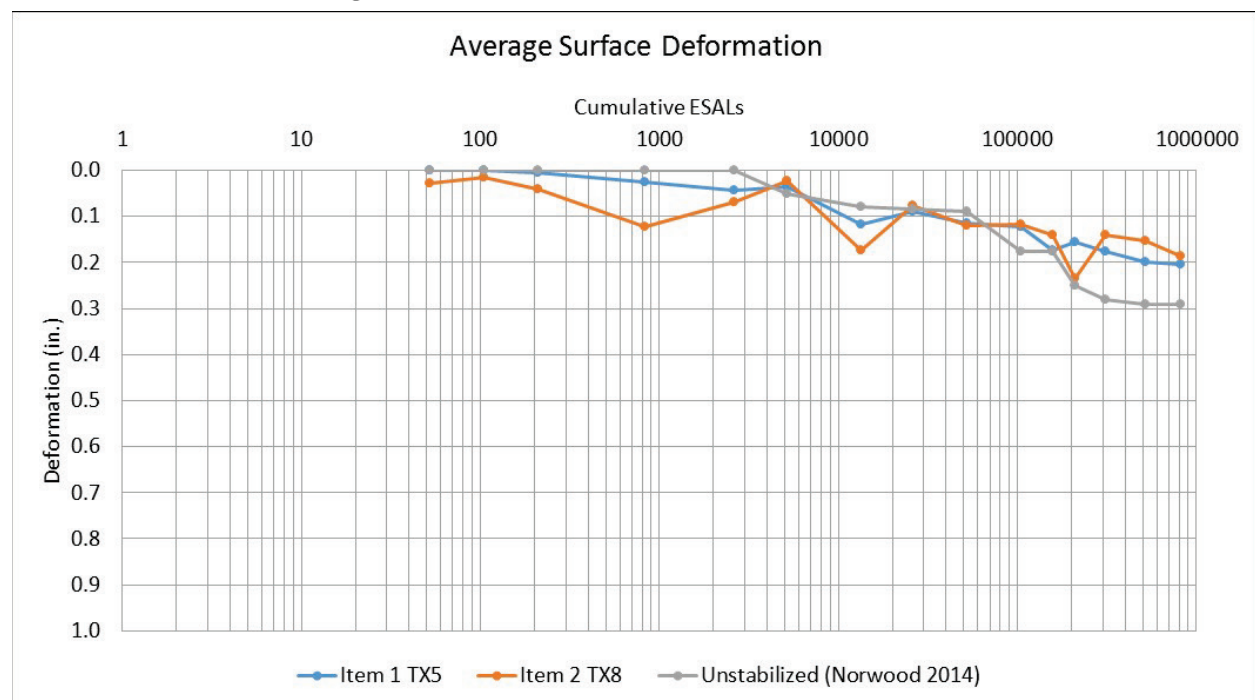
Table 5. Permanent surface deformation measurements.

Test Item	Pavement Structure	ESALS								
		54	108	216	865	27,040	54,080	162,240	324,480	811,200
Item 1	3-in. HMA 6-in. base stabilized, TX5	0.000	0.000	0.005	0.025	0.090	0.115	0.175	0.176	0.203
Item 2	3-in. HMA 6-in. base stabilized, TX8	0.027	0.016	0.040	0.124	0.077	0.121	0.141	0.140	0.187
Control	4-in. HMA 8-in. base unstabilized	0.000	0.000	0.000	0.000	0.085	0.090	0.175	0.280	0.290

7.2 Falling weight deflectometer

FWD data were collected at specified traffic intervals to monitor pavement structure stiffness. The ISM, which is the applied load divided by the measured load plate deflection, was used as the basis for comparison. FWD data were collected at Stations 12.5, 15, 20, 24, 28, and 37 at each designated traffic interval. Average as-built ISM values for Item 1 and Item 2 were 357 kips/in. and 410 kips/in., respectively. The average as-built ISM value for the control section was 521 kips/in. The increased stiffness of the unstabilized section was expected due to the additional 2 in. of base course and additional 1 in. of HMA. Change in ISM values with traffic for each test item are presented in Figure 21.

Figure 21. ISM change with traffic.
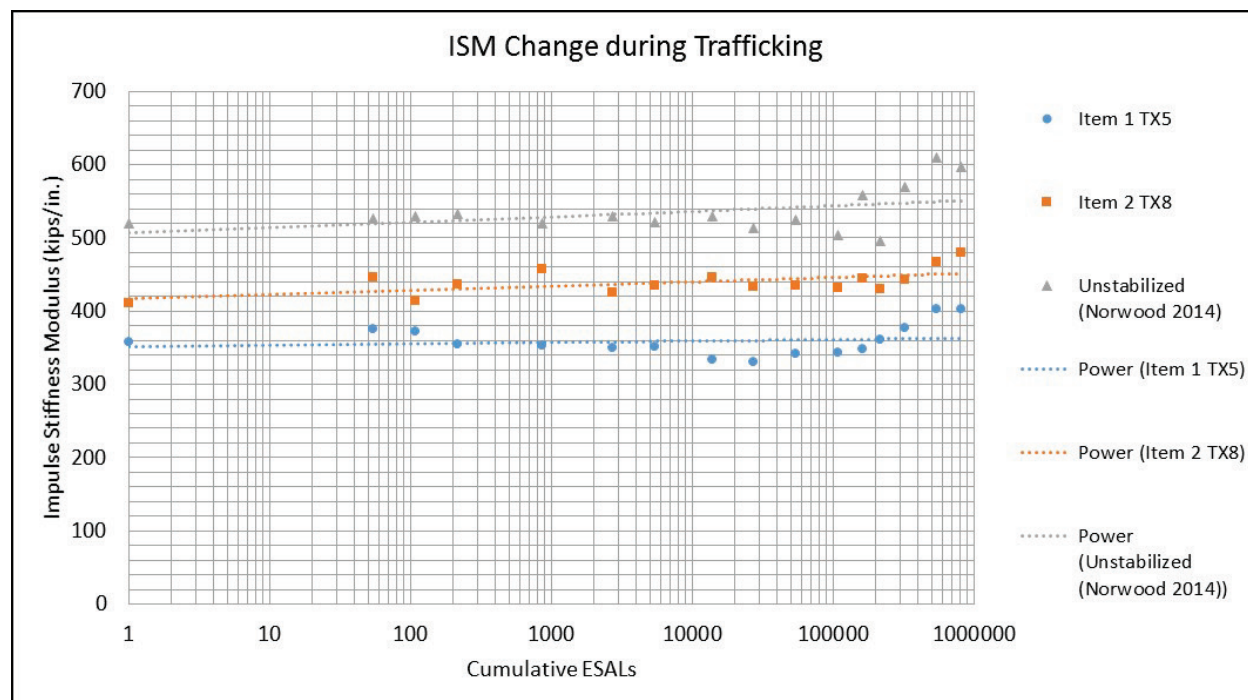
Under cumulative traffic loading, little change was observed in stiffness of the stabilized test items. A slight upward trend was noted at approximately 108,000 ESALS for each item trafficked that may have been a result of layer densification under sustained trafficking. The final ISM value for Item 1 was 402 kips/in., representing an increase of 45 kips/in. or approximately 13%. The final ISM value for Item 2 was 480 kips/in., representing an increase of 70 kips/in. or approximately 17%.

The Base Damage Index (BDI) was calculated as the difference between the measured deflection at the load plate (D1) and the measured deflection at a 12-in. offset (D2) to determine the relative stiffness of the aggregate base course. Figure 22 presents a plot of the calculated average BDI for each section at each traffic interval. The test results indicate the base stiffness values for Item 1 were lower than the unstabilized item throughout the duration of trafficking. The increased stiffness of the unstabilized section was expected due to the additional 2 in. of base course. Base stiffness values in Item 2 were found to be generally the same as those calculated for the unstabilized section.

Figure 22. Base damage index with traffic.
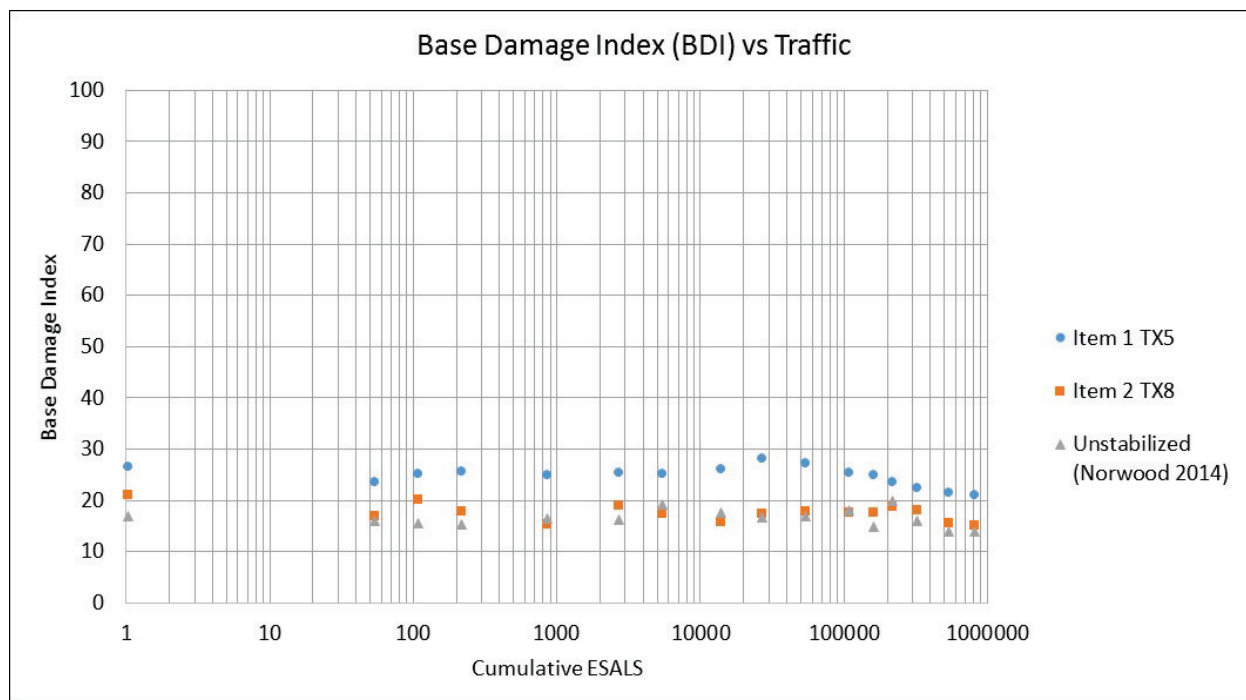
7.3 Earth pressure cells

Earth pressure cells were installed near the top of the base and near the top of the subgrade as described previously to collect dynamic stress response during trafficking. Locations and installation depths for each test item are presented in Table 6.

Predicted stresses within the pavement structure were calculated using the Pavement Engineering Utility Version 7 (PSEVEN). PSEVEN is a software tool developed at the ERDC for the analysis, design and evaluation of pavement structures. This tool implements pavement criteria and

procedures contained in Department of Defense Unified Facilities Criteria (UFC) 3-260-02. PSEVEN can be used for the analysis of flexible, rigid, unsurfaced, and mat-surfaced pavements. Required pavement thicknesses, allowable loads and allowable passes are computed for one or more ground vehicles and aircraft. PSEVEN is able to use vehicle geometrical and load data from the standard database supplied with the pavement engineering and design software package PCASE 2.09. Custom vehicles can be created and managed to add vehicles not included in the standard vehicles database. PSEVEN can also be used to design and analyze pavements using the layered elastic theory. Other features of PSEVEN include: frost calculations, minimum thickness of asphalt surfaces, and Aircraft Classification Numbers. The calculated stresses using PSEVEN are presented in Table 6 with the measured responses recorded at the beginning of the traffic testing.

Table 6. Predicted and measured earth pressure cell responses.

Location			As-built HMA Thickness (in.)	As-built Base Thickness (in.)	EPC depth from surface (in.)	Predicted Stress (psi)	Measured Stress (psi)	Measured vs. Predicted (psi)
Item 1	Station 15	Subgrade	3.72	5.52	11.24	7.7	6.3	-1.4
Item 1	Station 28	Subgrade	3.24	5.76	11.00	8.2	7.6	-0.6
Item 1	Station 20	Base	3.24	6.12	5.24	32.7	33.9	1.2
Item 1	Station 33	Base	3.12	5.64	5.12	32.7	30.5	-2.2
Item 2	Station 15	Subgrade	3.24	5.60	10.84	8.3	7.3	-1.0
Item 2	Station 28	Subgrade	3.24	5.88	11.12	8.1	13.0	4.9
Item 2	Station 20	Base	3.12	5.76	5.12	33.4	28.1	-5.3
Item 2	Station 33	Base	3.36	6.12	5.36	30.9	33.1	2.2

Dynamic response was measured for approximately 10 minutes of traffic after each designated data collection point to allow for HVS wheel wander and to account for any minor instrumentation misalignments during construction. The response data set were plotted and the maximum pressure reading was selected to represent the response at the designated traffic interval. Collecting data in this manner and selecting the maximum response assisted in ensuring the HVS wheel load was directly over the pressure cell(s). Earth pressure cell response data for the 10-minute test for each designated traffic interval are presented in Appendix A.

The maximum earth pressure cell response value from each 10-minute test were collected for the subgrade and base course. Changes in the EPC response values installed in the subgrade for Item 1 and Item 2 are shown in Figure 23. Changes in the EPC response values installed in the base course for Item 1 and Item 2 are shown in Figure 24.

Subgrade and base earth pressure cell response values in the stabilized sections were found to be generally equivalent to measured responses in the unstabilized section. A slight upward trend was observed in subgrade pressure with increasing traffic that may be due to densification and stiffening over time.

Figure 23. Peak subgrade earth pressure cell response during trafficking.
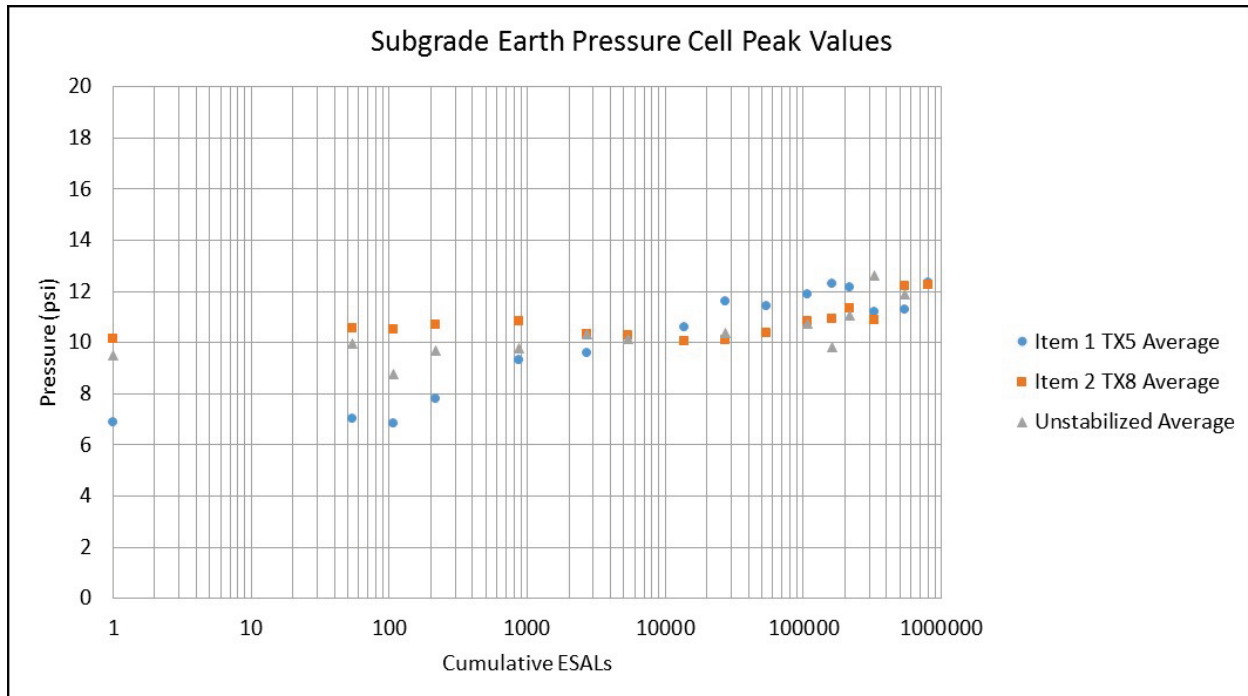
Figure 24. Peak base earth pressure cell response during trafficking.
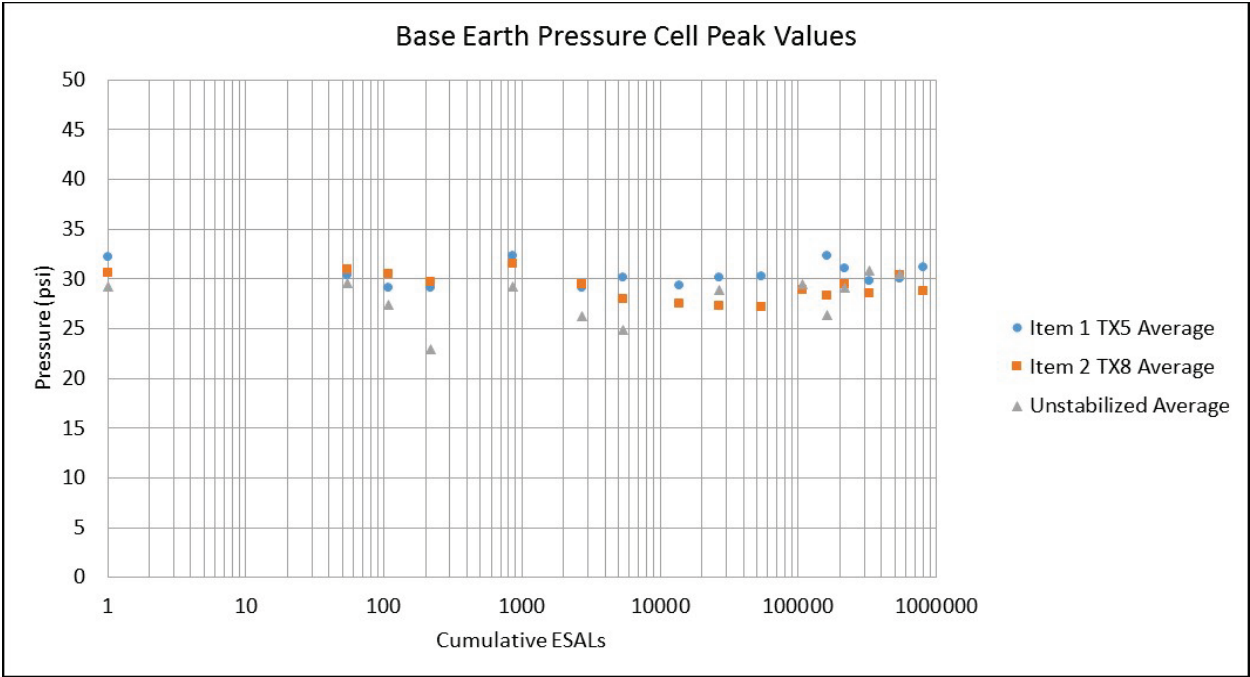
7.4 Single-depth deflectometers

Single-depth deflectometers were installed near the center of each test item (approximately Station 24). The gauges had a measurement range of ±2 inches and were installed at the top of the subgrade to give an estimate of deflections experienced by the subgrade. Single-depth deflectometer response data for the 10-minute test for each designated traffic interval are presented in Appendix B. Installation depths, predicted deflection values using PSEVEN, and measured deflection values for each test item are presented in Table 7.

Table 7. Predicted and measured single-depth deflectometer responses.

Location			As-built HMA Thickness (in.)	As-built Base Thickness (in.)	SDD depth from surface (in.)	Predicted deflection(in.)	Measured deflection (in.)	Measured vs. Predicted (in.)
Item 1	Station 24	Subgrade	3.12	5.88	11.00	0.029849	0.021309	-0.00854
Item 2	Station 24	Subgrade	3.12	5.76	10.88	0.030039	0.019779	-0.01026

Peak deflection values were obtained from the 10-minute test after each designated traffic interval. SDD peak value changes with traffic are shown in Figure 25. Measured deflections in the stabilized items were approximately ½ of that observed in the unstabilized section. This may be indicative of base stiffening due to aggregate interlock at the geogrid/aggregate interface.

Figure 25. Peak subgrade deflection during trafficking.
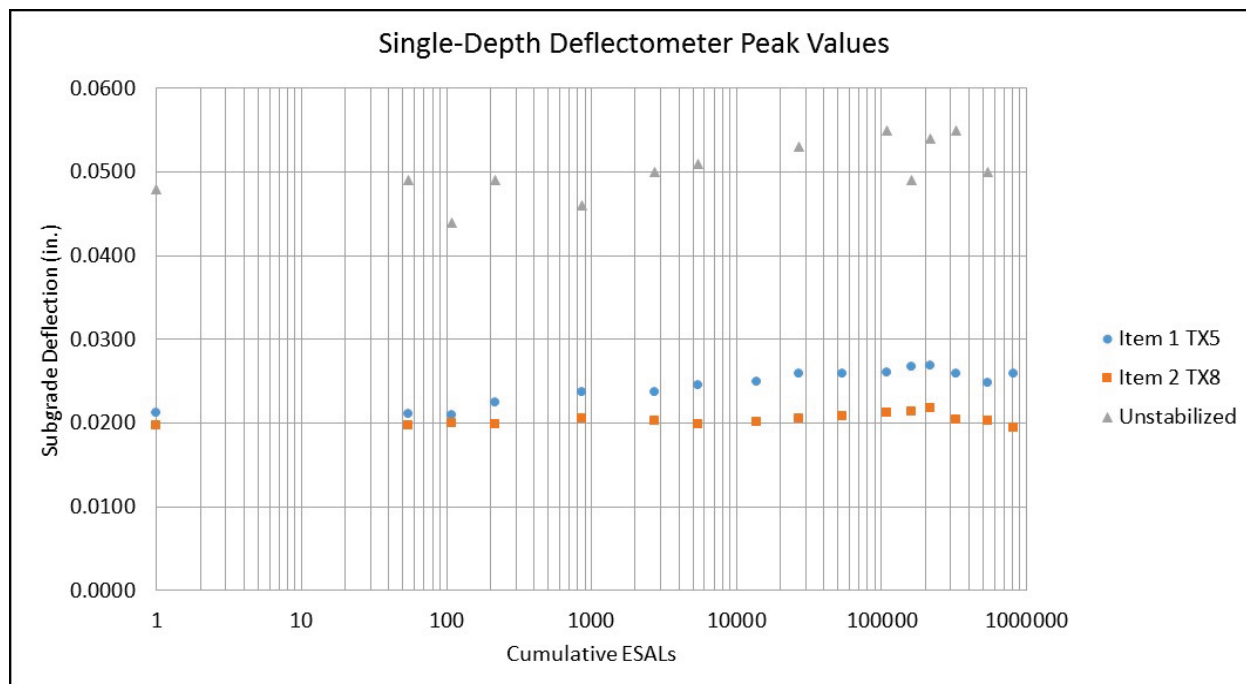
It is noted that the measured deflection values were lower than the PSEVEN predicted values in both test items, a possible indication of base course stiffening provided by the geogrids.

7.5 Asphalt strain gauges

Asphalt strain gauges (ASGs) were installed at Stations 12.5 and 37 in each item for this study. Two strain gauges were placed at each location with one oriented in the transverse direction and one oriented in the longitudinal direction. Response data were collected for a 10-minute interval at each designated data collection point, as with all other instrumentation. Dynamic asphalt strain response data are presented in Appendix C. Installation depths, predicted strain values, and measured strain values are presented in Table 8. Peak tensile strain response values are presented in Figure 26. It is noted that the measured peak tensile strain responses for the unstabilized section were much higher than that measured in the stabilized sections. This phenomena was not anticipated as higher peak strain values were expected with a 1 in. reduction in asphalt thickness. Different asphalt strain gauges were utilized for this study due to construction survivability concerns identified in previous work. It is believed the difference in gauge manufacturer could be a cause of the observed peak asphalt strain gauge responses. It is recommended caution be exercised when comparing these data.

Table 8. Predicted and measured asphalt strain gauge responses.

Location/Orientation			As-built HMA Thickness (in.)	As-built Base Thickness (in.)	ASG depth from surface (in.)	Predicted Microstrain (x 10⁻⁶ in./in.)	Measured Microstrain (x 10⁻⁶ in./in.)	Measured vs. Predicted (x 10⁻⁶ in./in.)
Item 1	Station 12.5	Transverse	3.36	5.94	3.36	216	158	-58
Item 1	Station 12.5	Longitudinal	3.36	5.94	3.36	263	250	-13
Item 1	Station 37	Transverse	2.88	5.88	2.88	228	167	-61
Item 1	Station 37	Longitudinal	2.88	5.88	2.88	269	260	-9
Item 2	Station 12.5	Transverse	3.30	5.58	3.30	222	146	-76
Item 2	Station 12.5	Longitudinal	3.30	5.58	3.30	269	175	-94
Item 2	Station 37	Transverse	3.36	6.24	3.36	213	153	-60
Item 2	Station 37	Longitudinal	3.36	6.24	3.36	259	143	-116

Figure 26. Peak asphalt tensile strain during trafficking.
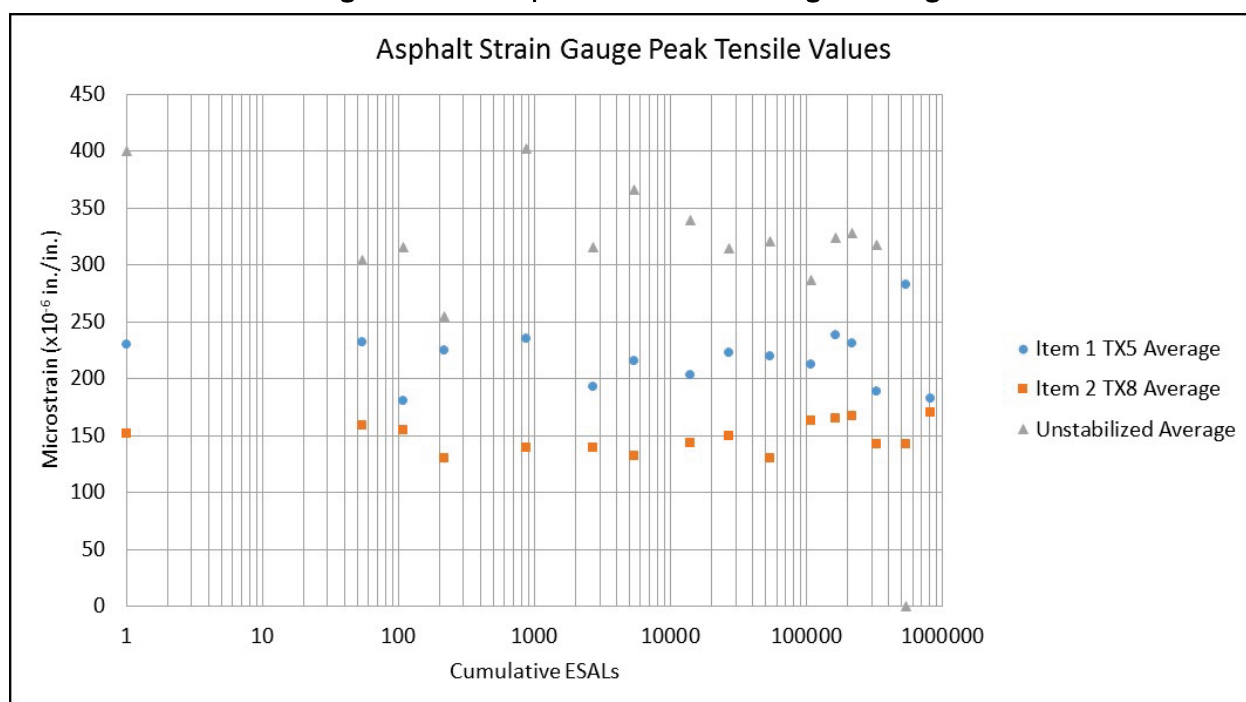
8 Post-Traffic Forensics

After the conclusion of traffic testing, each test item was trenched in two locations to investigate the method of pavement failure and characterize material properties. Trench locations were selected to correspond with the locations of minimum and maximum rutting in each item, as determined by review of both the rutting and surface deformation data. Each trench was approximately 3 ft wide and extended across the full traffic lane. Excavation began by saw cutting the HMA surface and carefully removing it to avoid disturbing the underlying base course. Measurements of each layer thickness were made at each trench location. In-field CBR tests, nuclear density measurements, and moisture content determinations were performed on each layer both within the wheel path and outside the wheel path. Results of tests performed in the trench locations for each item are shown in Table 9.

Table 9. Post-traffic physical data.

Property	Item 1 TX5 Stabilized Base	Item 2 TX8 Stabilized Base	Control 4-in. Nominal AC Unstabilized Base (Norwood and Tingle 2014)
CH Subgrade Properties			
Wet Density (pcf)	116.9	117.9	113.9
Dry Density (pcf)	89.1	89.1	88.6
Nuclear Moisture Content (%)	31.2	32.3	28.1
Oven-Dried Moisture (%)	32.7	32.3	30.1
In-Place CBR (%)	11.1	9.8	11.0
Crushed Limestone Properties			
Wet Density (pcf)	140.5	142.6	**
Dry Density (pcf)	136.7	138.7	**
Nuclear Moisture Content (%)	2.8	2.9	**
Oven-Dried Moisture (%)	2.2	1.7	**
In-Place CBR (%)	100+	100+	**

**Values not reported due to base disturbance during excavation

The test results indicate there was a minor increase in dry density of the subgrade, while the moisture content was relatively constant. A 2.2 pcf increase was measured in the base course dry density for Item 1 and a 1.5 pcf increase in base course density was measured for Item 2. Base course moisture contents for Items 1 and 2 decreased by 0.8% and 1.5%, respectively. An increase in subgrade CBR in the wheelpath was observed in both test sections, which was found to be consistent with observations made by Norwood and Tingle (2014).

The base course was carefully excavated to expose the underlying geogrid for visual inspection as shown in Figure 27 and Figure 28. Visual investigation revealed indentations were present in the subgrade of both test items (Figure 29 and Figure 30), indicating aggregate strike-through and interlock with the geogrid. Visual inspection indicated the geogrid was intact and no damage was sustained during construction or trafficking.

Visual inspection of the subsurface layers was performed to determine rutting behavior in each test section. The excavated areas are shown in Figure 31 through Figure 33 with orange markers used to delineate each layer interface. No significant permanent deformation was observed in the subgrade of both test items. Visual inspection indicated that the accumulated deformation was limited to the HMA and base-course layers.

Figure 27. Item 1 excavated geogrid.
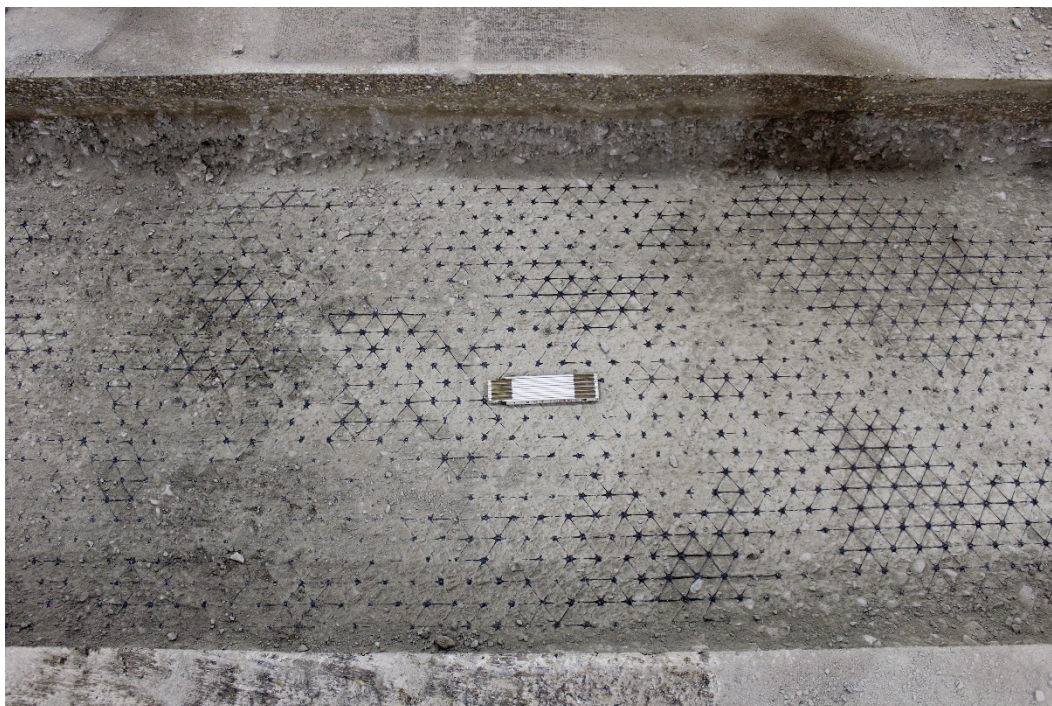
Figure 28. Item 2 excavated geogrid.
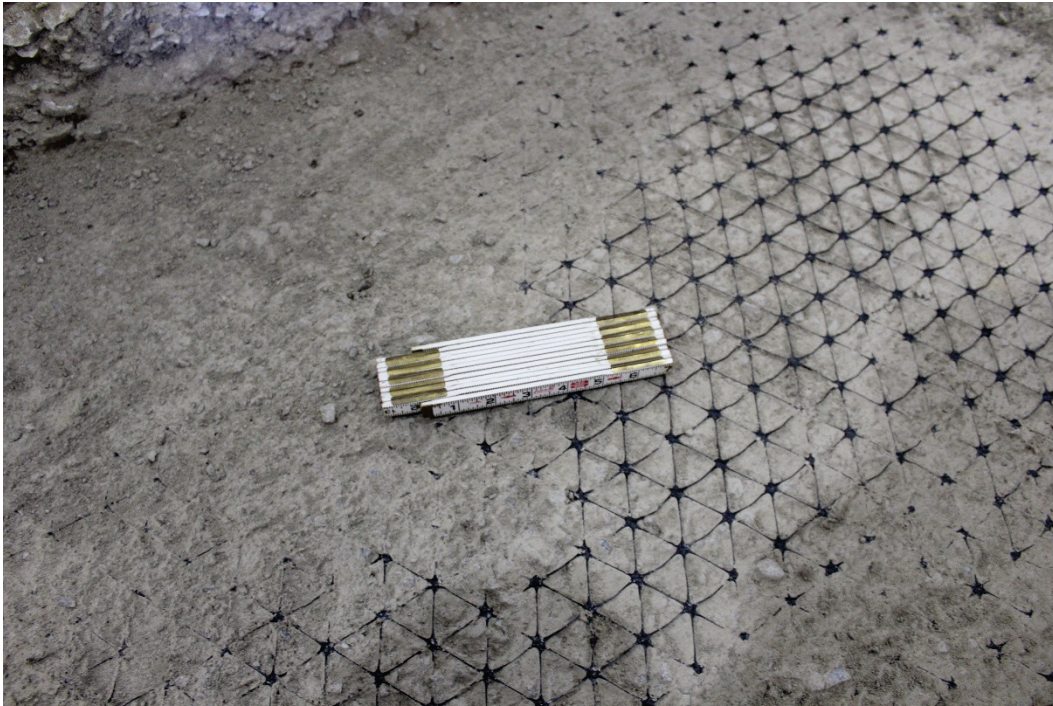
Figure 29. Item 1 indentations in subgrade.
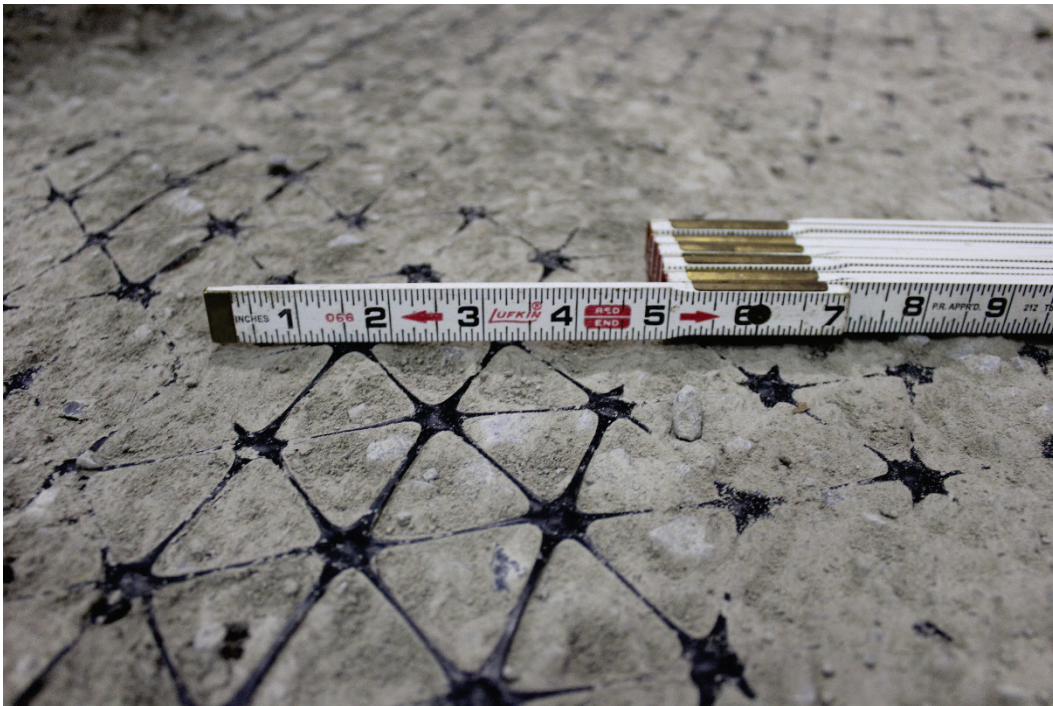
Figure 30. Item 2 indentations in subgrade.
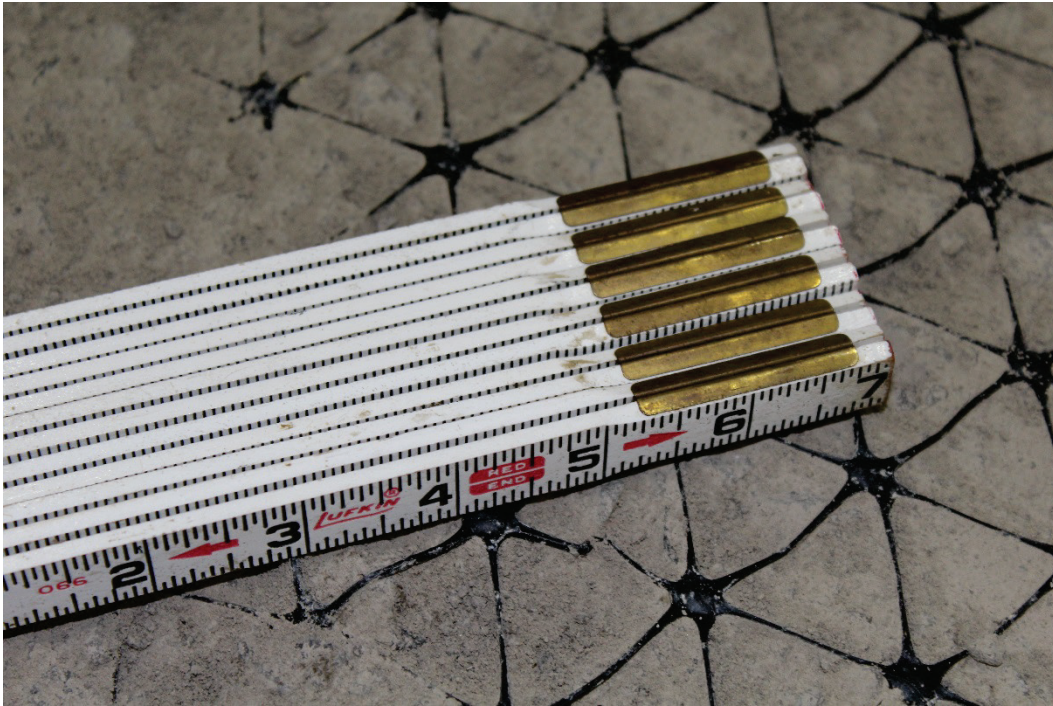
Figure 31. Post-traffic pavement cross section in Item 1.
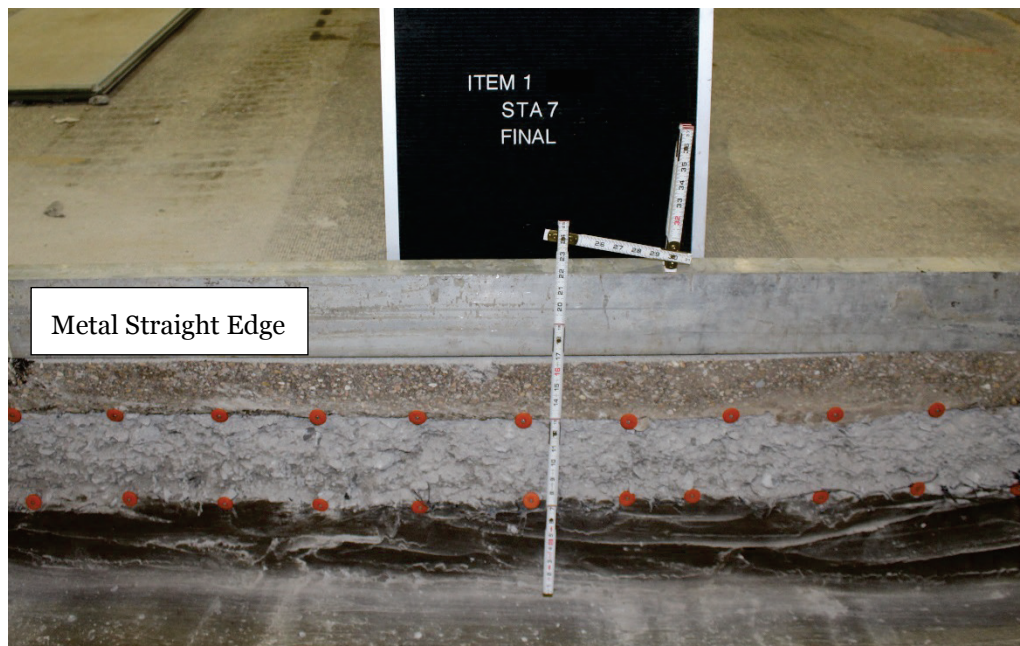
Figure 32. Post-traffic pavement cross section in Item 2.
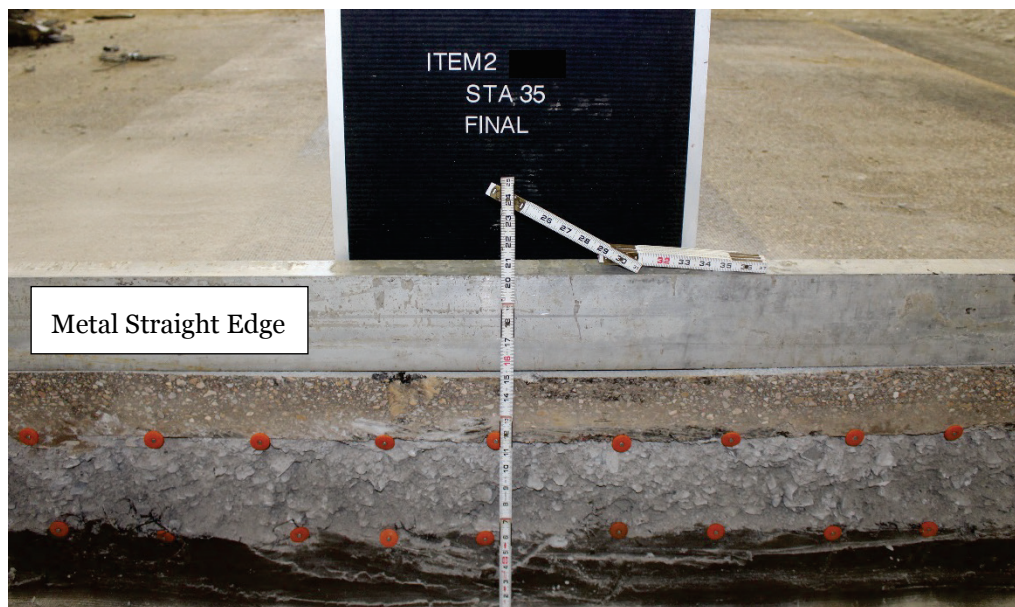
Figure 33. Post-traffic pavement cross section in unstabilized section.
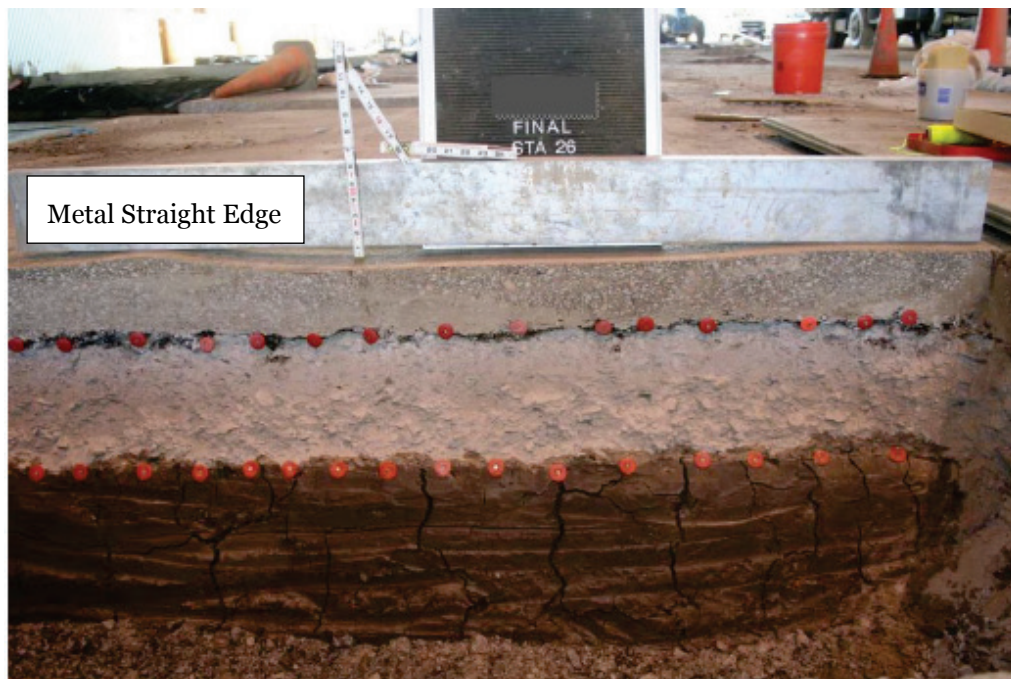
9 Analysis

In order to provide a measure of the performance benefit of each geogrid, an analysis was performed to calculate the effective structural number of the stabilized and unstabilized test items. The analysis results are discussed in the following paragraph.

In order to quantify the structural contribution of the geogrid, an effective structural number was calculated using the AASHTO Pavement Design Guide (1993). The effective structural number was calculated using the applied ESALs and the as-built layer thicknesses to compute the structural coefficient of the base-course layer. The effective structural coefficient provides a means of comparing test sections of varying layer thicknesses. Table 10 summarizes the effective base coefficient for each test section. A 90% reliability and an asphalt structural coefficient of 0.44 was selected for the calculations. The calculations indicate the different geogrid items had nearly the same effective structural coefficient, with both items displaying a significant increase in structural capacity over the unstabilized control item. It should be noted that none of the test sections reached the failure criteria (1 in. of rutting); therefore, the structural coefficients presented should be considered conservative estimates.

Table 10. Effective stabilized layer structural coefficient.

	Item 1	Item 2	Unstabilized
Design Base Thickness (in.)	6	6	8
Design Asphalt Thickness (in.)	3	3	4
Unstabilized Design Structural Number	2.16	2.16	2.88
As-Built Base Thickness (in.)	5.8	5.9	7.7
As-Built Asphalt Thickness (in.)	3.20	3.22	4
Unstabilized As-Built Structural Number	2.22	2.24	2.84
Passes to 1-in. rutting (ESALs)	811,200+	811,200+	811,200+
Effective Structural Number	3.08+	3.08+	3.08+
Effective Base Coefficient (1-in. rutting)	0.29+	0.28+	0.17+
Passes to ½-in. rutting (ESALs)	811,200+	811,200+	500,000
Effective Structural Number	3.08+	3.08+	2.86
Effective Base Coefficient (1/2-in. rutting)	0.29+	0.28+	0.14

10 Conclusions and Recommendations

A full-scale test section was constructed and trafficked to evaluate the performance of geogrid-stabilized highway pavements. The construction and traffic data were analyzed to assess both the relative performance of two multi-axial geogrid stabilized pavement sections and their performance as compared to an unstabilized pavement section from a previous related study (Norwood and Tingle 2014). The following conclusions are a result of this study:

1. The pavement test sections were constructed in a uniform manner with minor variability utilizing similar material and construction techniques used in previous geosynthetic stabilized test sections. The uniformity of construction and materials allow for meaningful comparisons between test sections to be made.
2. As-built ISM values for the stabilized items were found to be less than those for the unstabilized items, which was expected due to a reduced pavement section.
3. Subgrade and base earth pressure cell response values in the stabilized sections were found to be generally equivalent to measured responses in the unstabilized section. A slight upward trend was observed in subgrade pressure with increasing traffic that may be due to densification and stiffening over time.
4. Measured deflections in the stabilized items were approximately ½ of that observed in the unstabilized section. This may be indicative of base stiffening due to aggregate interlock at the geogrid/aggregate interface.
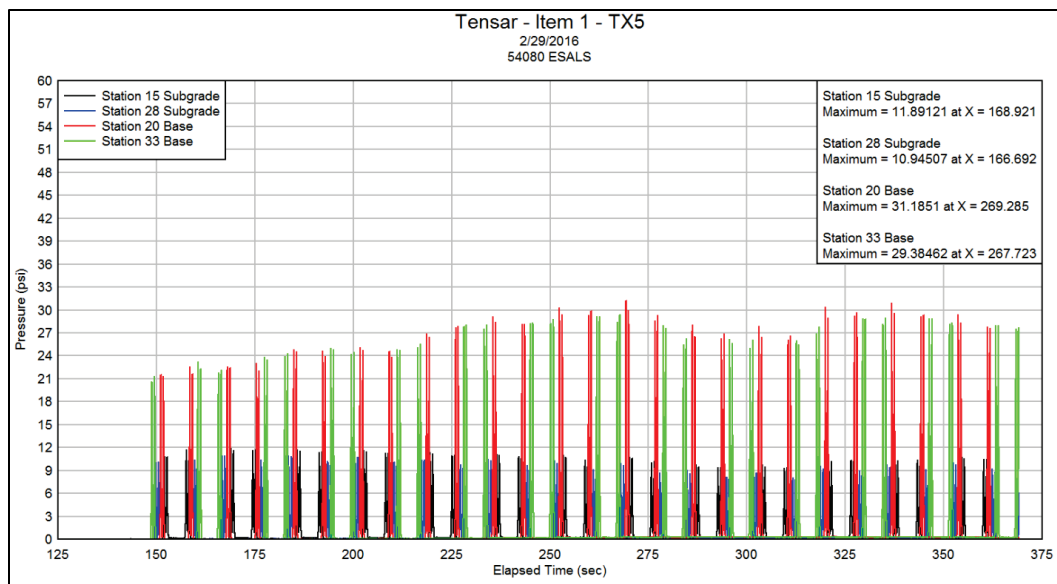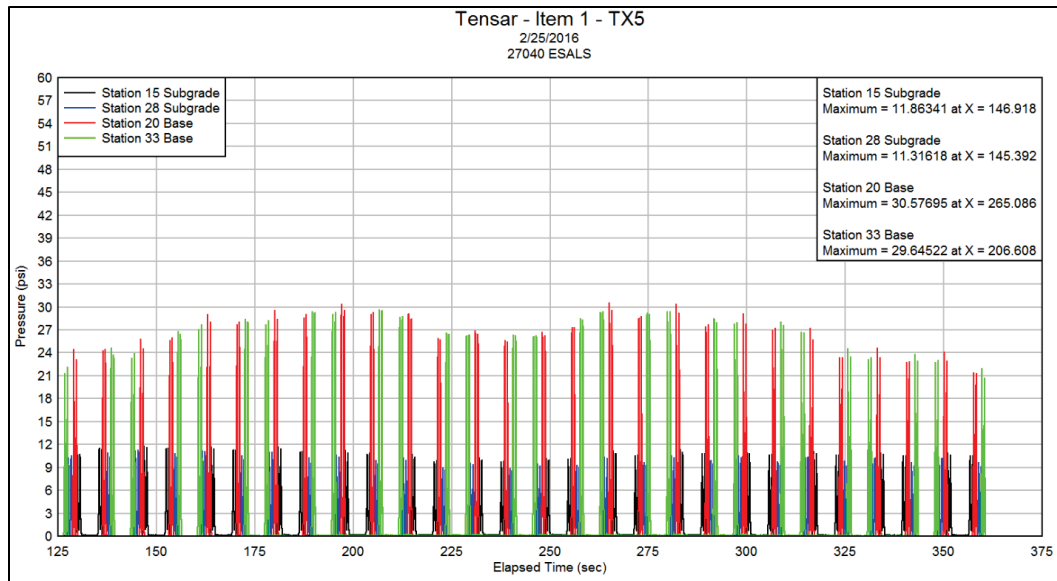5. Asphalt strain gauge response in the stabilized sections were less than that measured in the unstabilized section, however, differing gauge manufacturers were utilized. It is recommended that caution be exercised when comparing these data.
6. Both geogrid stabilized pavement sections improved the rutting resistance when compared to the unstabilized test section.
7. Minimal differences in rutting performance were observed between the two stabilized test sections.
8. The calculated effective base structural coefficient of the geogrid stabilized pavement sections was higher than the unstabilized structural coefficient, suggesting incorporation of a multi-axial geogrid in a flexible pavement base course provides a significant structural benefit.

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Appendix A: Earth Pressure Cell Responses

Tensar - Item 1 - TX5
2/18/2016
108 ESALS

Station 15 Subgrade
Maximum = 6.970796 at X = 46.148

Station 28 Subgrade
Maximum = 6.693975 at X = 72.931

Station 20 Base
Maximum = 31.59585 at X = 209.412

Station 33 Base
Maximum = 26.70098 at X = 139.906

Tensar - Item 1 - TX5
2/18/2016
216 ESALS

Station 15 Subgrade
Maximum = 7.929752 at X = 42.463

Station 28 Subgrade
Maximum = 7.714474 at X = 24.097

Station 20 Base
Maximum = 30.60808 at X = 193.291

Station 33 Base
Maximum = 27.54734 at X = 259.194

Tensar - Item 1 - TX5
2/19/2016
865 ESALS

Station 15 Subgrade
Maximum = 9.562976 at X = 792.944

Station 28 Subgrade
Maximum = 9.120089 at X = 790.083

Station 20 Base
Maximum = 33.6887 at X = 571.256

Station 33 Base
Maximum = 30.89671 at X = 586.65

Tensar - Item 1 - TX5
2/22/2016
2704 ESALS

Station 15 Subgrade
Maximum = 9.696201 at X = 6.249

Station 28 Subgrade
Maximum = 9.455651 at X = 14.614

Station 20 Base
Maximum = 30.24773 at X = 184.667

Station 33 Base
Maximum = 27.91925 at X = 183.135

Tensar - Item 1 - TX5
2/25/2016
27040 ESALS

Station 15 Subgrade
Maximum = 11.86341 at X = 146.918

Station 28 Subgrade
Maximum = 11.31618 at X = 145.392

Station 20 Base
Maximum = 30.57695 at X = 265.086

Station 33 Base
Maximum = 29.64522 at X = 206.608

Tensar - Item 1 - TX5
2/29/2016
54080 ESALS

Station 15 Subgrade
Maximum = 11.89121 at X = 168.921

Station 28 Subgrade
Maximum = 10.94507 at X = 166.692

Station 20 Base
Maximum = 31.1851 at X = 269.285

Station 33 Base
Maximum = 29.38462 at X = 267.723

Tensar - Item 1 - TX5
3/14/2016
216320 ESALS

Station 15 Subgrade
Maximum = 12.67548 at X = 157.125

Station 28 Subgrade
Maximum = 11.61412 at X = 133.127

Station 20 Base
Maximum = 32.4128 at X = 235.712

Station 33 Base
Maximum = 29.71222 at X = 233.508

Tensar - Item 1 - TX5
3/24/2016
324480 ESALS

Station 15 Subgrade
Maximum = 12.17812 at X = 689.04

Station 28 Subgrade
Maximum = 10.22769 at X = 698.054

Station 20 Base
Maximum = 30.78523 at X = 302.703

Station 33 Base
Maximum = 28.79721 at X = 320.986

Tensar - Item 1 - TX5
4/11/2016
540800 ESALS

Tensar - Item 1 - TX5
4/26/2016
811200 ESALS

Tensar - Item 2 - TX8
5/4/2016
108 ESALS

Station 15 Subgrade
Maximum = 7.839352 at X = 232.766

Station 28 Subgrade
Maximum = 13.15278 at X = 52.905

Station 20 Base
Maximum = 27.71254 at X = 205.2

Station 33 Base
Maximum = 33.27748 at X = 207.288

Tensar - Item 2 - TX8
5/4/2016
216 ESALS

Station 15 Subgrade
Maximum = 8.294756 at X = 225.496

Station 28 Subgrade
Maximum = 13.13309 at X = 232.5

Station 20 Base
Maximum = 27.88941 at X = 180.765

Station 33 Base
Maximum = 31.42566 at X = 402.474

Tensar - Item 2 - TX8
5/9/2016
5408 ESALS

Station 15 Subgrade
Maximum = 8.342489 at X = 227.35

Station 28 Subgrade
Maximum = 12.21374 at X = 252.068

Station 20 Base
Maximum = 25.74704 at X = 199.493

Station 33 Base
Maximum = 30.24109 at X = 190.472

Tensar - Item 2 - TX8
5/10/2016
13844 ESALS

Station 15 Subgrade
Maximum = 8.446397 at X = 8.695

Station 28 Subgrade
Maximum = 11.66171 at X = 235.023

Station 20 Base
Maximum = 23.09728 at X = 199.997

Station 33 Base
Maximum = 32.02885 at X = 202.162

Tensar - Item 2 - TX8
5/13/2016
27040 ESALS

Tensar - Item 2 - TX8
5/16/2016
54080 ESALS

Tensar - Item 2 - TX8
5/26/2016
216320 ESALS

Station 15 Subgrade
Maximum = 10.17831 at X = 183.256

Station 28 Subgrade
Maximum = 12.54176 at X = 185.37

Station 20 Base
Maximum = 26.97298 at X = 132.987

Station 33 Base
Maximum = 32.00956 at X = 123.34

Tensar - Item 2 - TX8
6/6/2016
324480 ESALS

Station 15 Subgrade
Maximum = 9.716694 at X = 123.911

Station 28 Subgrade
Maximum = 12.01856 at X = 114.25

Station 20 Base
Maximum = 26.16246 at X = 231.538

Station 33 Base
Maximum = 30.88056 at X = 221.291

Tensar - Item 2 - TX8
6/21/2016
540800 ESALS

Station 15 Subgrade
Maximum = 10.26092 at X = 19.327

Station 28 Subgrade
Maximum = 14.18673 at X = 295.38

Station 20 Base
Maximum = 27.83896 at X = 243.759

Station 33 Base
Maximum = 32.93721 at X = 233.538

Tensar - Item 2 - TX8
7/14/2016
811200 ESALS

Station 15 Subgrade
Maximum = 9.812598 at X = 232.259

Station 28 Subgrade
Maximum = 14.69856 at X = 223.762

Station 20 Base
Maximum = 26.47968 at X = 65.552

Station 33 Base
Maximum = 31.07915 at X = 55.428

Appendix B: Single-Depth Deflectometer Responses

Tensar - Item 2 - TX8
5/2/2016
0 ESALS

Tensar - Item 2 - TX8
5/3/2016
54 ESALS

Tensar - Item 2 - TX8
5/5/2016
865 ESALS

Station 24 Subgrade
Maximum = 0.02061123 at X = 243.738

Tensar - Item 2 - TX8
5/6/2016
2704 ESALS

Station 24 Subgrade
Maximum = 0.02025055 at X = 436.258

Tensar - Item 2 - TX8
5/9/2016
5408 ESALS

Tensar - Item 2 - TX8
5/10/2016
13844 ESALS

Tensar - Item 2 - TX8
5/13/2016
27040 ESALS

Station 24 Subgrade
Station 24 Subgrade
Maximum = 0.02062603 at X = 168.225

Tensar - Item 2 - TX8
5/16/2016
54080 ESALS

Station 24 Subgrade
Station 24 Subgrade
Maximum = 0.02079174 at X = 86.056

Tensar - Item 2 - TX8
5/19/2016
108160 ESALS

Station 24 Subgrade
Maximum = 0.02122622 at X = 108.746

Tensar - Item 2 - TX8
5/23/2016
162240 ESALS

Station 24 Subgrade
Maximum = 0.02140924 at X = 462.719

Tensar - Item 2 - TX8
5/26/2016
216320 ESALS

Station 24 Subgrade

Station 24 Subgrade
Maximum = 0.02173984 at X = 218.371

Tensar - Item 2 - TX8
6/6/2016
324480 ESALS

Station 24 Subgrade

Station 24 Subgrade
Maximum = 0.02048583 at X = 114.226

Tensar - Item 2 - TX8
6/21/2016
540800 ESALS

Station 24 Subgrade
Station 24 Subgrade
Maximum = 0.0203502 at X = 76.571

Tensar - Item 2 - TX8
7/14/2016
811200 ESALS
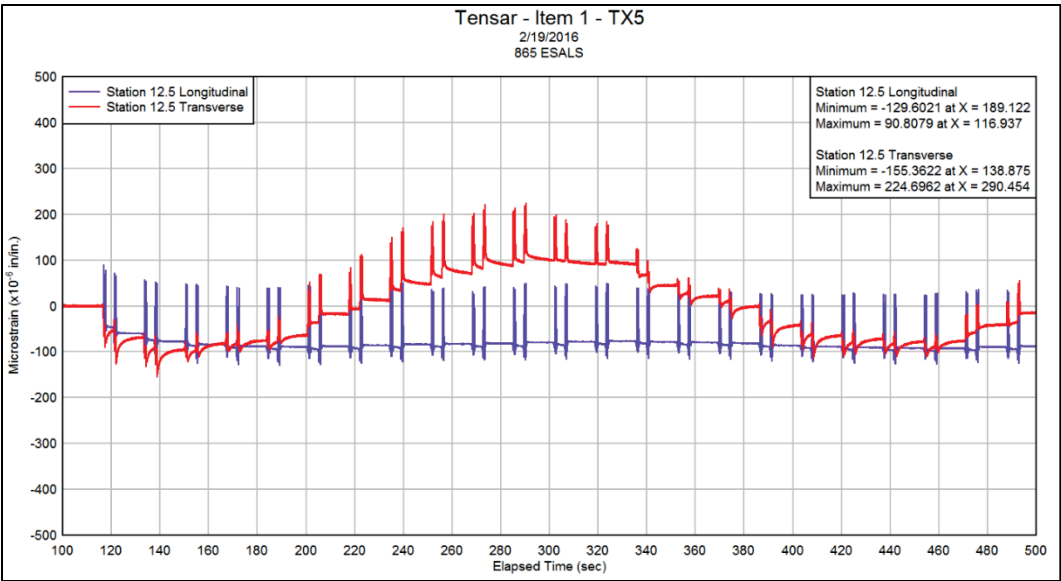
Station 24 Subgrade
Station 24 Subgrade
Maximum = 0.01948394 at X = 200.115

Appendix C: Asphalt Strain Gauge Responses

Tensar - Item 1 - TX5
2/19/2016
865 ESALS

Tensar - Item 1 - TX5
2/19/2016
865 ESALS

Tensar - Item 1 - TX5
2/23/2016
5408 ESALS

Station 12.5 Longitudinal
Minimum = -132.3524 at X = 358.48
Maximum = 83.7076 at X = 47.723

Station 12.5 Transverse
Minimum = -119.6769 at X = 324.94
Maximum = 227.3172 at X = 201.096

Tensar - Item 1 - TX5
2/23/2016
5408 ESALS

Station 37 Longitudinal
Minimum = -203.5242 at X = 502.598
Maximum = 109.5898 at X = 56.305

Station 37 Transverse
Minimum = -183.0508 at X = 315.753
Maximum = 191.2772 at X = 196.662

Tensar - Item 1 - TX5
2/24/2016
13844 ESALS

Station 12.5 Longitudinal
Minimum = -145.5171 at X = 426.641
Maximum = 89.42293 at X = 185.434

Station 12.5 Transverse
Minimum = -131.1851 at X = 241.304
Maximum = 169.6645 at X = 359.164

Tensar - Item 1 - TX5
2/24/2016
13844 ESALS

Station 37 Longitudinal
Minimum = -132.9859 at X = 384.076
Maximum = 103.1211 at X = 261.396

Station 37 Transverse
Minimum = -236.2785 at X = 215.296
Maximum = 114.8135 at X = 350.281

Tensar - Item 1 - TX5
2/29/2016
54080 ESALS

Station 12.5 Longitudinal
Station 12.5 Transverse

Station 12.5 Longitudinal
Minimum = -20.61147 at X = 157.472
Maximum = 189.9985 at X = 326.498

Station 12.5 Transverse
Minimum = -84.49219 at X = 174.957
Maximum = 249.7848 at X = 309.654

Tensar - Item 1 - TX5
2/29/2016
54080 ESALS

Station 37 Longitudinal
Station 37 Transverse

Station 37 Longitudinal
Minimum = -187.9218 at X = 368.247
Maximum = 111.0252 at X = 161.211

Station 37 Transverse
Minimum = -189.3906 at X = 165.758
Maximum = 177.6764 at X = 317.68

Tensar - Item 1 - TX5
3/4/2016
108160 ESALS

Station 12.5 Longitudinal
Minimum = -213.4544 at X = 405.805
Maximum = 90.11562 at X = 11.701

Station 12.5 Transverse
Minimum = -139.3246 at X = 389.451
Maximum = 156.0745 at X = 253.698

Tensar - Item 1 - TX5
3/4/2016
108160 ESALS

Station 37 Longitudinal
Minimum = -210.5909 at X = 447.883
Maximum = 110.5141 at X = 20.261

Station 37 Transverse
Minimum = -192.6896 at X = 109.408
Maximum = 173.6514 at X = 244.816

Tensar - Item 1 - TX5
3/9/2016
162240 ESALS

Station 12.5 Longitudinal
Minimum = -73.05752 at X = 344.478
Maximum = 161.1625 at X = 141.484

Station 12.5 Transverse
Minimum = -75.4103 at X = 311.183
Maximum = 261.7737 at X = 175.392

Tensar - Item 1 - TX5
3/9/2016
162240 ESALS

Station 37 Longitudinal
Minimum = -97.43435 at X = 200.25
Maximum = 177.1756 at X = 145.003

Station 37 Transverse
Minimum = -204.548 at X = 318.924
Maximum = 215.527 at X = 200.329

Tensar - Item 2 - TX8
5/2/2016
0 ESALS

Station 12.5 Longitudinal
Minimum = -88.17012 at X = 440.973
Maximum = 93.02688 at X = 368.872

Station 12.5 Transverse
Minimum = -106.6132 at X = 525.619
Maximum = 136.4645 at X = 407.558

Tensar - Item 2 - TX8
5/2/2016
0 ESALS

Station 37 Longitudinal
Minimum = -109.7801 at X = 580.266
Maximum = 62.39992 at X = 364.395

Station 37 Transverse
Minimum = -80.81604 at X = 516.675
Maximum = 166.8 at X = 381.842

Tensar - Item 2 - TX8
5/3/2016
54 ESALS

Station 12.5 Longitudinal
Minimum = -88.6382 at X = 176.838
Maximum = 102.3638 at X = 54.046

Station 12.5 Transverse
Minimum = -123.9937 at X = 261.453
Maximum = 137.6149 at X = 126.434

Tensar - Item 2 - TX8
5/3/2016
54 ESALS

Station 37 Longitudinal
Minimum = -54.2915 at X = 265.37
Maximum = 98.9885 at X = 49.547

Station 37 Transverse
Minimum = -39.34484 at X = 265.651
Maximum = 180.3142 at X = 147.496

Tensar - Item 2 - TX8
5/4/2016
216 ESALS

Station 12.5 Longitudinal
Minimum = -52.54475 at X = 196.603
Maximum = 130.8312 at X = 314.717

Station 12.5 Transverse
Minimum = -99.46039 at X = 297.933
Maximum = 133.0803 at X = 398.997

Tensar - Item 2 - TX8
5/4/2016
216 ESALS

Station 37 Longitudinal
Minimum = -54.73859 at X = 386.106
Maximum = 105.8114 at X = 170.479

Station 37 Transverse
Minimum = -91.85503 at X = 305.838
Maximum = 126.715 at X = 187.892

Tensar - Item 2 - TX8
5/5/2016
865 ESALS

Station 12.5 Longitudinal
Minimum = -56.81895 at X = 207.677
Maximum = 137.45 at X = 89.008

Station 12.5 Transverse
Minimum = -143.408 at X = 38.971
Maximum = 149.085 at X = 186.536

Tensar - Item 2 - TX8
5/5/2016
865 ESALS

Station 37 Longitudinal
Minimum = -71.50454 at X = 232.652
Maximum = 97.39546 at X = 93.126

Station 37 Transverse
Minimum = -121.8204 at X = 30.021
Maximum = 129.4266 at X = 165.226

Tensar - Item 2 - TX8
5/6/2016
2704 ESALS

Station 12.5 Longitudinal
Minimum = -80.48669 at X = 316.184
Maximum = 119.5923 at X = 450.335

Station 12.5 Transverse
Minimum = -140.5106 at X = 114.861
Maximum = 150.5287 at X = 547.39

Tensar - Item 2 - TX8
5/6/2016
2704 ESALS

Station 37 Longitudinal
Minimum = -127.2841 at X = 559.82
Maximum = 80.12592 at X = 21.386

Station 37 Transverse
Minimum = -126.3996 at X = 391.664
Maximum = 125.5734 at X = 256.541

Tensar - Item 2 - TX8
5/9/2016
5408 ESALS

Station 12.5 Longitudinal
Minimum = -121.0747 at X = 232.198
Maximum = 103.6973 at X = 41.686

Station 12.5 Transverse
Minimum = -105.8343 at X = 46.8
Maximum = 126.7067 at X = 181.813

Tensar - Item 2 - TX8
5/9/2016
5408 ESALS

Station 37 Longitudinal
Minimum = -137.5215 at X = 257.181
Maximum = 65.16846 at X = 37.327

Station 37 Transverse
Minimum = -90.44268 at X = 54.736
Maximum = 126.6753 at X = 186.075

Tensar - Item 2 - TX8
5/10/2016
13844 ESALS

Station 12.5 Longitudinal
Minimum = -110.5154 at X = 232.054
Maximum = 113.8926 at X = 8.685

Station 12.5 Transverse
Minimum = -132.0272 at X = 47.282
Maximum = 150.6554 at X = 215.438

Tensar - Item 2 - TX8
5/10/2016
13844 ESALS

Station 37 Longitudinal
Minimum = -136.2484 at X = 223.269
Maximum = 75.16155 at X = 4.356

Station 37 Transverse
Minimum = -132.4756 at X = 55.197
Maximum = 131.9993 at X = 189.06

Tensar - Item 2 - TX8
5/16/2016
54080 ESALS

Station 12.5 Longitudinal
Minimum = -110.0965 at X = 235.557
Maximum = 119.3955 at X = 15.997

Station 12.5 Transverse
Minimum = -144.789 at X = 84.051
Maximum = 116.8196 at X = 235.819

Tensar - Item 2 - TX8
5/16/2016
54080 ESALS

Station 37 Longitudinal
Minimum = -64.93127 at X = 239.793
Maximum = 113.4187 at X = 138.202

Station 37 Transverse
Minimum = -111.2123 at X = 109.277
Maximum = 114.9827 at X = 223.253

Tensar - Item 2 - TX8
6/6/2016
324480 ESALS

Tensar - Item 2 - TX8
6/6/2016
324480 ESALS

Tensar - Item 2 - TX8
6/21/2016
540800 ESALS

Station 12.5 Longitudinal
Minimum = -154.4878 at X = 292.472
Maximum = 109.1382 at X = 19.322

Station 12.5 Transverse
Minimum = -131.8218 at X = 91.38
Maximum = 127.243 at X = 242.352

Tensar - Item 2 - TX8
6/21/2016
540800 ESALS

Station 37 Longitudinal
Minimum = -67.81013 at X = 266.9
Maximum = 121.0699 at X = 132.254

Station 37 Transverse
Minimum = -101.6976 at X = 82.499
Maximum = 130.6684 at X = 229.827

Tensar - Item 2 - TX8
7/14/2016
811200 ESALS

Tensar - Item 2 - TX8
7/14/2016
811200 ESALS

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14. ABSTRACT

The U.S. Army Engineer Research and Development Center (ERDC) constructed a full-scale test section to evaluate the performance of geogrid-stabilized highway pavements. The test section consisted of two pavements composed of 3 in. of hot mix asphalt over a crushed limestone base course over a 6 CBR subgrade. One section was stabilized with Tensar's TX5 multi-axial geogrid, and the second section was stabilized with a developmental geogrid referred to as TX8. Each section was instrumented with moisture probes, pore-water pressure probes, temperature probes, earth pressure cells, and asphalt strain gauges. Simulated truck traffic was applied using a heavy vehicle simulator (HVS) to evaluate the rutting performance and collect instrumentation response data. This report summarizes the material characterization and construction techniques, instrumentation response, and performance response of each section under simulated truck traffic. Comparisons are drawn to previous sections constructed and trafficked at ERDC without geogrids utilizing similar construction techniques and material properties.

15. SUBJECT TERMS		
Geogrid	Pavements – Testing	Geogrids
Base stabilization	Effective structural coefficient	Soil stabilization
Pavements – Performance	Pavements, Flexible	
	Heavy vehicle simulator	

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